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ORDINARY MEETING.

30 November, 1937.

MAURICE FITZGERALD WILSON, Vice-President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Stanley Dymock Canvin, B.Sc. (Eng.) (*Lond.*).
Ernest Walter Cook.
Sydney Eric Mallaby Firth, B.Sc. (Eng.) (*Lond.*).
William Arthur Harrison, M.Eng. (*Liverpool*).
Neil Brodie Henderson.

And had admitted as

Students.

Ronald Anstey.
Donald Laurence Bates, B.A. (*Cantab.*).
Joseph Hubert Beard.
Harry George Bill.
James Lowe Blake.
John Ralph Bower.
Peter Stewart Bowis.
Roy Beauchamp Brown.
John Burley.
Henry Herman Collette, B.Sc. (*Lond.*).
Terence Eldon Crowley, B.Sc. (*Leeds*).
Desmond Durkin.
David Ifor Emlyn Evans.
Basil William Fenton.
Maurice Samuel Franses, B.Sc. (Eng.) (*Lond.*).
Richard Thomas Gladwin.
Alexander Gray.
William Compton Hall.
George Allan Fenn Hally, B.A., (*Cantab.*).
John Huckstep.
William Joseph Hilton Hulme.
Charles Drysdale Smith Hume.
Deryk Thomas Hurlstone.
William Ralph Latham.
John Charles Edwin Lowcock.
Colin Pitblado MacDiarmid.
Ronald Keith McWilliam.

George Bruce Marriott.
Hugh McKinty Montgomery, B.Sc. (*Belfast*).
Ivan Geoffrey Moore, B.Sc. (*Leeds*).
Henry Olivier, B.Sc. (*Cape Town*).
David Paton.
William Jeffrey Randall.
George Norman Ray.
Rupert Barnwell Reed.
Richard James Rees.
Albert Ernest Ridings, B.Sc. (*Edin.*).
John Rioch.
Jack Charles Sandford.
Shirley Anthony Vincent, B.Sc. (Eng.) (*Lond.*).
Geoffrey James Whitehouse.
Colin Williams, B.Eng. (*Liverpool*).

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of The Institution were accorded to the Author.

Paper No. 5088.

"The Design and Operation of the Coleshill Sewage-Disposal Works of the Birmingham Tame and Rea District Drainage Board."†

By Frank Charles Vokes, B.Sc. (Eng.), Assoc. M. Inst. C.E.

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Preliminary Considerations.

The development of a hitherto sparsely-populated area to the south-east of Birmingham, lying at a comparatively low level, necessitated, in 1934, the construction of a trunk sewer $7\frac{1}{2}$ miles long, large enough to convey the sewage from an eventual population of 335,000. It necessitated also the construction of sewage-purification works (*Fig. 1*, p. 324) to treat the sewage from an existing population of 50,000, the layout being such as to allow the works to be extended from time to time as the contributing population increases up to a maximum of 335,000.

An area of land alongside the river Tame was available as a site for the works. At this point on the river the sewage could be discharged after treatment without pumping, provided that the treatment was carried out in tanks, with resulting economy of head, and not on bacteria-beds.

Character of Sewage to be Treated.

It was estimated that the sewage would be chiefly of a domestic character, and would be readily capable of complete treatment by

† Correspondence on this Paper can be accepted until the 15th April, 1938.—Acting Sec. Inst. C.E.

means of activated sludge. However, as for some years the sewage from a small population would be conveyed by a comparatively large sewer, it would be retained therein for so long a period that its effective treatment would at times be somewhat difficult. These

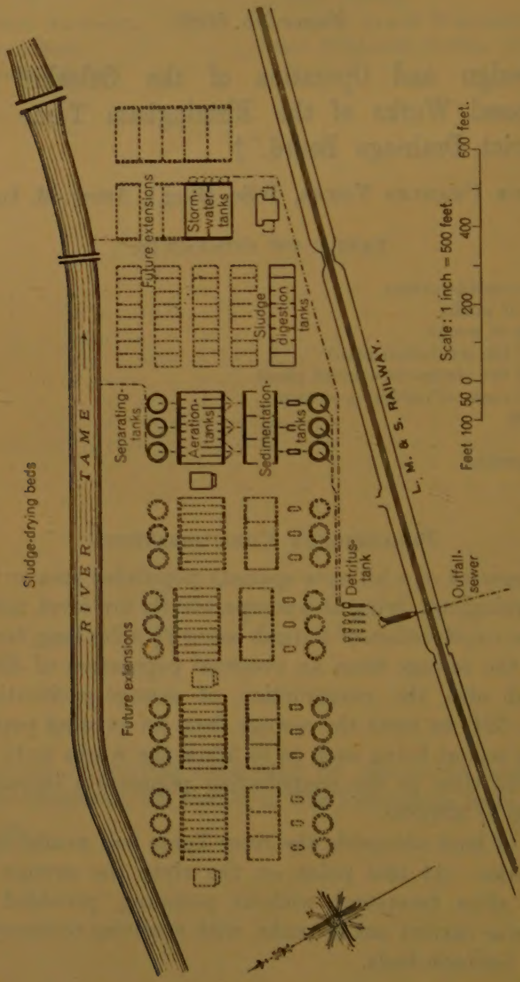


Fig. 1.

PLAN OF WORKS.

circumstances would at first be accentuated by the tipping into the trunk sewer of large quantities of cesspool-waste from districts not yet provided with sewers.

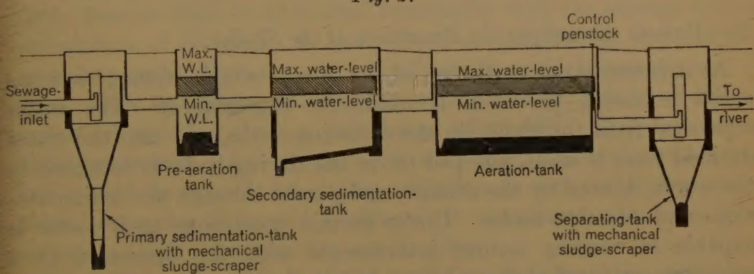
The extent to which the sewage is likely to alter in future may be judged by studying the recent history of the Birmingham Tame and Rea District Drainage Board's Yardley works. The average strength

of the crude sewage in the year 1925, as indicated by the amount of oxygen absorbed from potassium permanganate in 4 hours, was 6·73 parts per 100,000, but the sewage now contains considerable trade waste and may be described as "strong." The average strength of the crude sewage at the Yardley works during 1935 was 13·4 parts per 100,000, and in dry weather during the months of January and June the strength was 19·4 and 18·6 parts per 100,000 respectively.

The Process selected for the Purification of the Sewage.

The works include aeration-tanks for the treatment of the sedimented sewage by means of activated sludge. In the design of the plant provision has been made for the improvement of the sewage to the utmost practicable extent before it reaches the oxidation-plant. In order that the sensitive activated sludge shall not be

Fig. 2.



METHOD OF BALANCING RATE OF FLOW THROUGH PURIFICATION-PLANT.

adversely affected at times, precautions have been taken to shield the oxidation-plant from the effects of violent changes in the character of the sewage, as well as from large fluctuations in the rate of flow.

To prevent the oxidation-plant receiving sedimented sewage of an unnecessarily difficult character, sedimentation-tanks of ample capacity have been provided, thus ensuring effective separation of the sludge, and averaging of the strength of the sewage.

Large fluctuations in the rate of flow through the oxidation-plant have been prevented by the provision of a higher and a lower effluent-weir in the secondary sedimentation-tanks and also in the aeration-tanks, as shown in *Fig. 2*, thus enabling the plant to retain and give further treatment to the stronger sewage received during peak flow at about midday, and to discharge it during the period of minimum and usually weaker flow at night.

The sewage, before entering the secondary sedimentation-tanks, receives activated-sludge treatment for a short period, the beneficial

effect of which continues whilst the activated sludge is being separated from the sewage in the secondary sedimentation-tanks. For this purpose use is made of that portion of the activated sludge circulating in the aeration- and separating-tanks which is surplus to requirements. This surplus is in suitable condition, and no use would otherwise be made of its valuable properties.

The secondary sedimentation-tanks are completely emptied periodically and cleaned out, thus preventing fermenting sludge on the walls or floor interfering with the tank-effluent.

The pumps for returning activated sludge from the separating-tanks to the aeration-tanks are of ample capacity, thus making it possible to dilute a difficult influent with an increased quantity of effluent when necessary.

Flexibility in the aeration-plant has been ensured by dividing it into several units, so that the time of retention may be varied within wide limits if necessary.

The Process selected for the Treatment of the Sludge.

As it is necessary to dispose of the sewage-sludge without causing aerial nuisance, provision is made for digesting it. The water separated from the sludge in the digestion-tanks, and also the water drained from it when pumped on to the drying-beds, is returned to the sewer, diluted by the sewage, and passed through the sedimentation- and aeration-tanks. Under certain conditions such water is capable of causing serious interference with the operation of a sensitive activated-sludge plant. It has, however, been found that if the sludge is digested in two stages, all water drawn from primary tanks being passed through the secondary digestion-tanks also, the water can be treated satisfactorily and without interfering with the operation of the plant, provided that the rate is limited so as to ensure effective dilution. At the Coleshill works the sludge is digested in two stages.

The control of the process has been facilitated by heating the sludge in the primary tanks, the rate of digestion in winter being thus made equal to that in summer. This provision has allowed the digestion-tanks to be made of much smaller capacity.

The Source selected for the Supply of Power.

In selecting a suitable source of power-supply for the activated-sludge plant the following considerations were taken into account:—

(1) The power required for driving the air-compressors used in connection with the activated-sludge plant would be considerable and constant throughout the 24 hours.

(2) There would be sufficient gas evolved from the digesting sludge to generate this amount of power and supply waste heat from the engines for heating the sludge.

After careful consideration it was decided to collect the gas evolved from the sludge in the primary digestion-tanks, to utilize it in internal-combustion engines for the generation of electricity, and to utilize the waste heat in the cooling water and in the exhaust-gases for heating the digesting sludge.

Description of Works.

It will be seen from the general plan (*Fig. 1*, p. 324) that the river Tame passes across the 120-acre site acquired for the sewage-purification works. As already mentioned, it was necessary to provide for treating the sewage from an existing population of 50,000, and to lay out the site in such a way that the works could be extended from time to time so as to deal eventually with the sewage from a population of 335,000. It was, therefore, decided to lay out five complete units of plant, only one of which would be constructed at first. This first unit was designed to deal with 30 gallons of sewage per head per day from a population of 67,000. The unit would therefore receive a dry-weather flow of 2,000,000 gallons per 24 hours, but would be capable of giving oxidation as well as settlement to three times this flow, and settlement in storm-water tanks only to a further three times the dry-weather flow.

In planning the layout of the works on the site the purification-units were arranged to the south of the river, the land to the north being reserved as a sludge-drying area. The storm-water tanks were arranged to the east of the site, a 54-inch diameter cast-iron main being laid at the commencement. This is large enough to convey the storm-water from the future population of 335,000.

On emerging at the outfall to the works, the sewage passes through a coarse screen consisting of bars with 3½-inch openings, cleaned by hand. The sewage then enters a detritus-tank provided with three parallel channels, only one being used when the dry-weather flow is passing at a rate of 1 foot per second. When six times the dry-weather flow is passing through all three channels the water is at a higher level and the rate of flow does not exceed 1¼ foot per second. Detritus deposited in the side channels finds its way to the centre and deeper channel, from which it is lifted by an electrically-operated travelling dredger.

The sewage then passes between twin storm-water weirs having a total length of 30 feet. After parting with the storm-water the rate of flow of the sewage to the purification-plant is measured by means

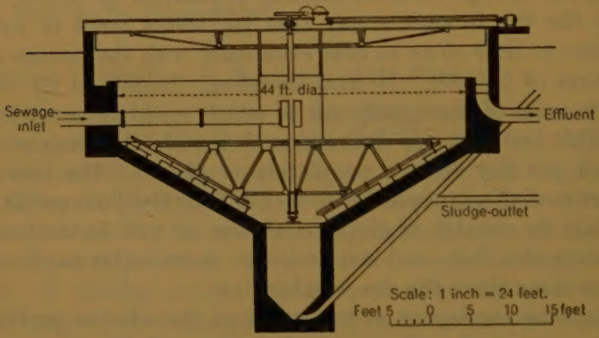
of a venturi meter which records up to three times the dry-weather flow.

Sewage-Purification Plant.

The purification-unit is arranged in three parallel lines. Each line consists of a primary sedimentation-tank, a pre-aeration tank, a secondary sedimentation-tank, an aeration-tank in three separate parallel bays, and a separating tank. The flow to the aeration-tank is measured by a venturi indicator.

Each of the three primary sedimentation-tanks is 44 feet in diameter (*Fig. 3*). The sewage enters at the centre and the effluent flows over a peripheral weir. The floor slopes towards the centre

Fig. 3.



PRIMARY SEDIMENTATION-TANK.

at an angle of 30 degrees to the horizontal. At the centre is a well 7 feet in diameter and 10 feet deep into which the sludge is delivered by an electrically-driven scraper operated occasionally. These tanks have a capacity of 4 hours' dry-weather flow.

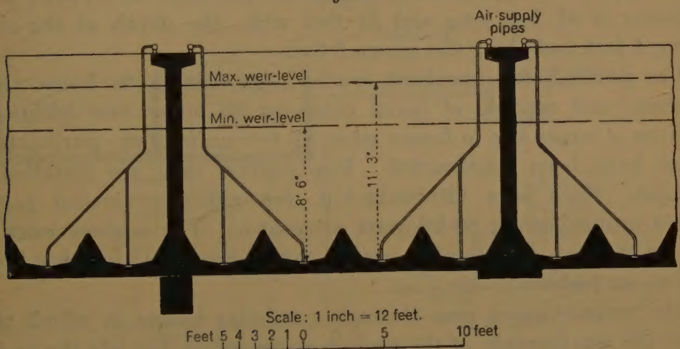
The pre-aeration tanks, three in number, are provided with compressed-air-diffusers, and receive the surplus sludge from the aeration-plant, with the aid of which they give treatment to the partly-sedimented sewage. They have a minimum capacity of ½ hour's dry-weather flow.

The three secondary sedimentation-tanks are rectangular in plan. Each is provided with a weir at the inlet end and a higher and a lower weir at the outlet end. By the use of the latter weirs the rate of flow from the tanks can be kept constant despite variations in the rate of flow to the tanks, the capacity varying from a minimum of 8·6 hours' to a maximum of 11·7 hours' dry-weather flow. When

emptied for cleaning the light sludge deposited on the sloping floor is pushed to the centre channel by hand.

The aeration-tanks (*Fig. 4*) comprise altogether nine separate parallel sections. Each is 20 feet wide, 110 feet long, and is provided with four lines of 6-inch diffusers set at the bottom of V-shaped troughs. The diffusers consist of detachable porous tiles fixed into cast-iron trays. Two weirs are provided at the effluent end, thus enabling the capacity of the tanks to be varied from a minimum of 12·3 hours' to a maximum of 16·4 hours' dry-weather flow. The depth of water over the diffusers varies from a minimum of 8 feet 6 inches to a maximum of 11 feet 3 inches. For the supply of

Fig. 4.



Cross-Section of Aeration-Tank.

compressed air to the aeration-tanks, rotary air-compressors of the crescent type with sliding blades are used.

Each of the three separating tanks is 44 feet in diameter, and is provided with an electrically-driven sludge-scraper which is in continuous operation. The water enters at the centre and the effluent flows over a peripheral weir. The floor slopes towards the centre at an angle of 30 degrees to the horizontal. These tanks have a capacity of 4 hours' dry-weather flow. When passing 3 times the dry-weather flow the maximum rate of upward flow is limited to 8·8 feet per hour.

Sludge-Treatment Plant.

For the treatment of the crude sludge, the digestion-tanks are arranged in one block of eight parallel rectangular tanks, each of which is 61 feet 4 inches long, 31 feet 4 inches wide, and 16 feet deep. Five of these are used as primary tanks, digestion of the sludge

being completed in the three secondary tanks. The total capacity provided is equivalent to 3¼ cubic feet per head of population. The total time of retention of the digesting sludge is approximately 12 weeks. Arrangements are provided for conveying surplus water from the secondary digestion-tanks to the outfall-sewer entering the works.

After digestion the sludge is pumped to the sludge-drying area. Each of the drying beds has an area of ½ acre. The total area provided is 4 acres, equivalent to an allowance of 3½ persons per square yard. Part of the dried sludge is sold to farmers and the remainder is tipped on a portion of the site.

Sheet-steel gas-collectors are provided on the five primary digestion-tanks, and are designed to float upon the sludge. The steel employed contains from 0·35 to 0·5 per cent. of copper; each collector is 61 feet long and 31 feet wide, the depth at the sides being 4 feet and along the crown 5 feet.

The power-house has been made large enough to house three engines, and capable of being extended to house two additional engines of larger size in future when all five units of the purification-plant have been constructed. Two engines only are installed at present. Each is a 120-brake-h.p. four-cylinder vertical engine direct-coupled to an 80-kilowatt alternator. The engines work on sludge gas, but, in case of necessity, can be converted in a few hours to run on fuel-oil.

The exhaust-gases pass through a tubular heater in which they raise the temperature of the cooling water which has left the engine cylinders (*Fig. 5*). This water passes through a tubular exchanger, in which it gives up its heat to alkaline water drawn from the sludge in the secondary digestion-tanks. The alkaline water is then pumped to the primary digestion-tanks, where it heats the incoming crude sludge and also corrects its *p*H value.

Construction of Works.

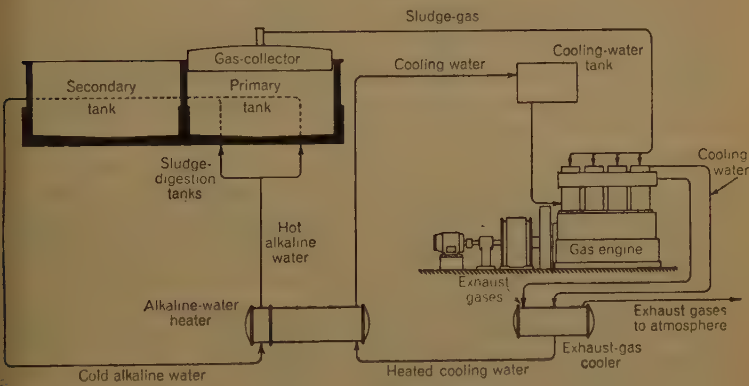
Before the construction of the Coleshill works was commenced by the Drainage Board, the course of the river Tame had been straightened, widened, and deepened by the Corporation of Birmingham. The land adjoining the banks of the old river course had previously been liable to floods. In spite of the improvement which had since been made in the river course, it was necessary to allow for a considerably higher future flood-level, although these conditions would not occur until complete development of a very large area of the watershed had taken place. Under these circumstances it was necessary to build up the walls of the tanks and the pathways around them to a level above the future flood-level.

The site was composed of a fairly loose gravel overlying marl. The excavation for all tanks was taken out to a batter, and a complete system of underdrainage was laid and connected to three deep sumps, from which water was pumped night and day during the progress of the work.

The primary sedimentation-tanks (*Fig. 3*, p. 328) were constructed of concrete without reinforcement, and were made sufficiently heavy to prevent flotation when empty.

In the secondary sedimentation-tanks, which are of rectangular form, 24-inch cut-off walls extending downwards 18 inches into the marl were constructed beneath all walls. Bituminous sheeting was

Fig. 5.



UTILIZATION OF WASTE HEAT.

laid on these cut-off walls, thus preventing them from acting as an integral part of the tank walls above. The floors were constructed of concrete 6 inches thick without reinforcement. Holes 4 inches in diameter were left in the floor at intervals for the purpose of relieving water-pressure beneath an empty tank. The cut-off walls prevent any considerable quantity of water passing beneath the tank walls. Above normal water-level the tank walls are perforated, thus preventing the walls being subjected to an unnecessarily high water-pressure in time of flood. The tank walls, which are 14 feet 6 inches high, are of inverted ⊤ form and were constructed of reinforced concrete. In order to prevent any tendency to slide on the water-logged marl, reinforced-concrete struts were provided at floor-level extending right across the three tanks. The storm-water tanks are of similar construction, the walls being 15 feet 6 inches high.

In the aeration-tanks the vertical walls are of reinforced concrete 13 feet 3 inches high and 12 inches thick. To withstand the horizontal pressure they are supported at the bottom by the floor and at the top by horizontal beams with horizontal reinforced-concrete struts.

The sludge-digestion tanks are of rectangular form and have walls 16 feet high constructed as cantilevers from the reinforced-concrete floor.

Operation of the Oxidation-Plant.

The oxidation-plant was put into operation in October, 1934. During the first year the mode of operating the plant was varied slightly from time to time in order to determine the best means of dealing with the varying character of the sewage.

At first a comparatively small period of retention in the aeration-tanks was allowed, but it was found difficult to maintain the activated sludge in a satisfactory condition. An increase in the time of retention made a distinct improvement.

During the first half of 1935, when a considerable number of houses were still connected to cesspools, the presence in the sewage of cesspool-waste prevented the activated sludge attaining its best condition. Improvement effected in the condition of the sludge was, at times, accompanied by some difficulty in controlling its tendency to rise and pass away over the weirs of the separating tanks. Slightly decreasing the time of retention in the aeration-tanks overcame this tendency of the sludge to rise. When, however, the time of retention was decreased too much, filamentous growths developed in the sludge and caused a tendency to bulking. For the purpose of settling an activated sludge so light in character and of such high organic content, it was found distinctly advantageous to utilize separating-tanks designed to give the minimum time of retention consistent with the desired low rate of upward flow; this object was assisted by the use of mechanical sludge-scrapers operating continuously.

Owing to the constantly-recurring changes in the composition and rate of flow of the sewage, the activated sludge was liable to change in character at short notice, and due vigilance had to be observed in order that the control of the plant might be modified from time to time without delay, so as to prevent the activated sludge losing its valuable properties.

In order to make the sludge settle more readily, and at the same time to improve the effluent, the weight of the sludge was increased by feeding into the aeration-tanks a small proportion of marl mixed up in water. The results have been described by Messrs. H. C.

Whitehead, M. Inst. C.E., and F. R. O'Shaughnessy, F.I.C.[1] The Authors state that " Normally, the sludge in this plant is of good quality, and reasonably dense so that the behaviour of the plant may be said to be satisfactory. Marl was added in the form of a suspension to the sludge entering the aeration chamber with the result that the effluent from the plant was brilliant in appearance, and the good settling qualities of the sludge were improved and a denser sludge obtained."

A typical analysis of the crude sewage so far received at the Coleshill works is as follows :—

	Parts per 100,000 by weight.
Suspended solids	17·2
Free ammonia	3·36
Albuminoid ammonia	0·75
Chlorides (as chlorine)	6·0
Oxygen absorbed in 4 hours at 80° F. from N/80 $KMnO_4$:—	
(*a*) Sample containing solids in suspension	4·88
(*b*) Sample after 2 hours' quiescent sedimentation	3·04
Biological oxygen-demand	22·8
(Dissolved oxygen taken up by sample from Birmingham tap water when incubated for 5 days at 65° F.)	
Alkalinity (expressed in terms of $CaCO_3$)	26·0

The operation of the plant is indicated by the typical results given in Table I; the samples analysed were taken between 8 a.m. on the 3rd and 8 a.m. on the 4th February, 1936.

TABLE I.

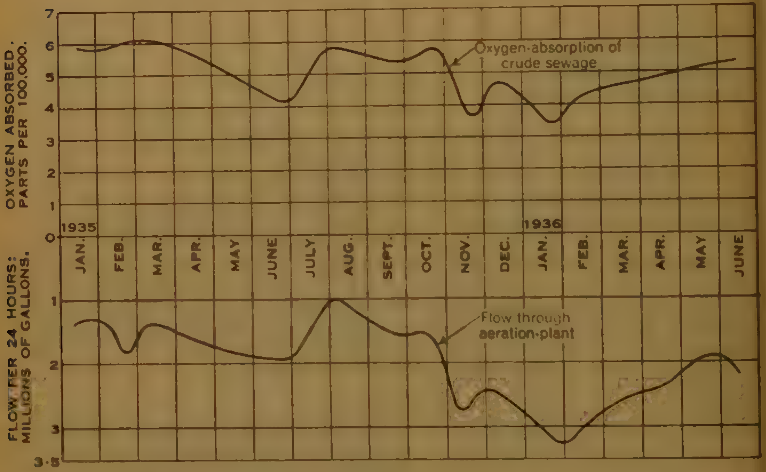
	Parts per 100,000 by weight.				
	Crude sewage.	Primary sedimentation-tank effluent.	Pre-aeration tank effluent.	Secondary sedimentation-tank effluent.	Final effluent.
Suspended solids	17·7	5·6	11·0	2·8	Trace.
Free ammonia	3·26	3·19	3·12	2·51	2·87
Albuminoid ammonia	0·68	0·67	0·39	0·43	0·13
Chlorides (as chlorine)	7·2	8·0	7·6	5·6	6·0
Oxidized nitrogen	—	—	—	—	0·15
Oxygen absorbed in 4 hours at 80° F. from N/80 $KMnO_4$:					
(*a*) Sample containing solids in suspension	4·56	4·12	4·00	2·32	0·68
(*b*) Sample after 2 hours' quiescent sedimentation	3·28	3·32	2·88	2·08	0·64

The average biological oxygen-demand of the final effluent during March, April, and May, 1936, was 0·76, and the average percentage saturation of dissolved oxygen of the final effluent during March, April, and May, 1936, was 40·8.

[1] "Improving the Efficiency of Activated Sludge." *The Surveyor*, vol. lxxxix (1936), p. 407.

The effect of pre-aeration on subsequent purification in the secondary sedimentation-tanks is shown by the following tests. A sample of effluent from each of the three sedimentation-tanks was taken every twenty minutes from 10.30 a.m. to 2.30 p.m. on the 10th June, 1936. This was repeated on the 17th and 24th June. On each occasion a different one of the three pre-aeration tanks was shut down for the purpose. The corresponding secondary sedimentation-tank had been cleaned at a time intermediate between

Fig. 6.



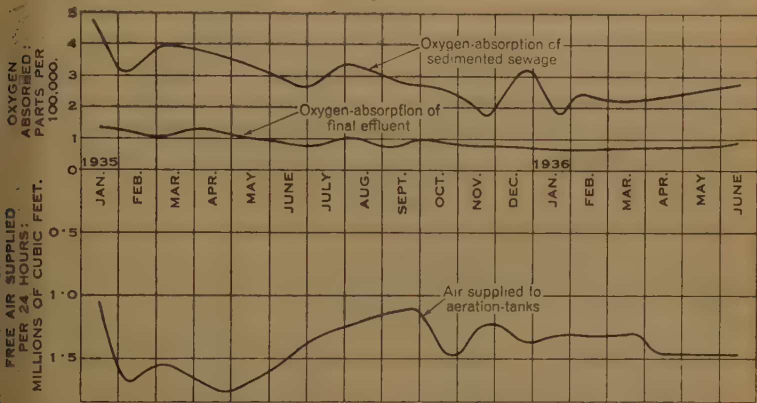
the cleaning of the tanks which were receiving pre-aerated sewage. Typical analyses of the effluents from the secondary sedimentation-tanks were as follows :—

	Parts per 100,000 by weight.	
	Without pre-aeration.	With pre-aeration.
Albuminoid ammonia	0·48	0·40
Oxygen absorbed in 4 hours at 80° F. from N/80 $KMnO_4$	3·03	2·29
Biological oxygen-demand in 5 days . .	10·70	7·03

The curves in *Fig. 6* show how the strength and the flow of sewage have varied over the 18 months during which the plant has been in operation. To enable the effect of dilution on the strength of the sewage to be seen more readily, the lower curve is plotted downwards. For the sake of convenience this method has been employed in the case of other curves which follow.

From *Fig. 7* may be seen the extent of the purification effected in the oxidation-plant and the amount of air which had to be compressed for this purpose.

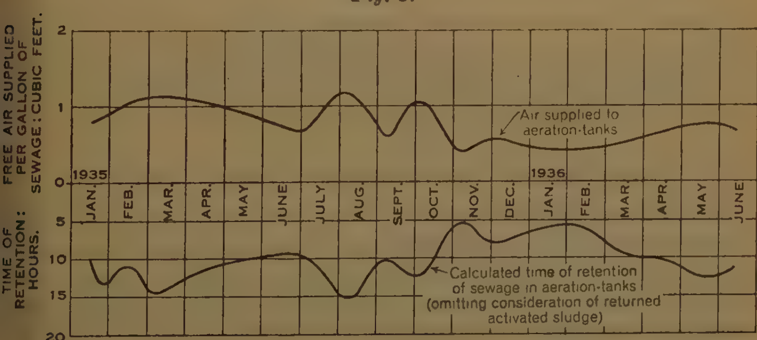
Fig. 7.



The size of the oxidation-plant utilized is indicated by the time-of-retention curve in *Fig. 8*, which also shows the amount of air used per gallon of sewage.

As both strength and flow of sewage were varying continually, it is useful to have a measure of the total impurities in the sedimented sewage entering the oxidation-plant. This is indicated in *Fig. 9* (p.

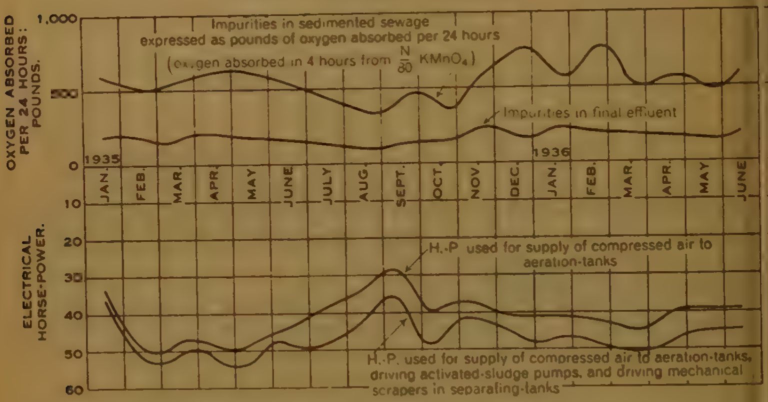
Fig. 8.



336), which also shows the variation in the amount of power required for driving the air-compressors. The quantity of activated sludge returned to the aeration-tanks was approximately 576,000 gallons per day, and the quantity of surplus activated sludge passed into the pre-aeration tanks 72,000 gallons per day. A sample drawn from the aeration-tanks showed approximately 10 per cent. of settled sludge after 1 hour or 7 per cent after 18 hours' settlement.

The value of the preliminary treatment in pre-aeration tanks by means of the surplus activated sludge, followed by additional sedimentation and balancing of the flow, is shown in the curves in *Fig. 10* taken from an earlier Paper by Messrs. H. C. Whitehead and F. R. O'Shaughnessy.[1] Curve A shows the total impurities in the crude sewage. The impurities remaining after the sewage has received 4 hours' sedimentation are shown in Curve B. There follows activated-sludge treatment in pre-aeration tanks for a period of ½ hour, the effect of which is continued during retention in the secondary sedimentation- and balancing-tanks for a further 8·6 to

Fig. 9.



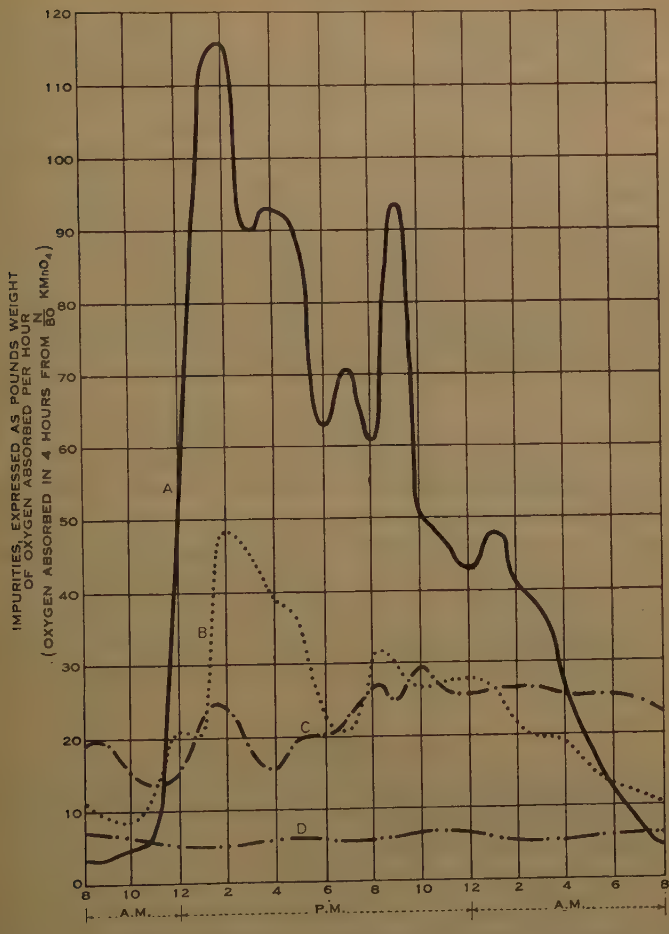
11·7 hours. Curve C shows the impurities left to be removed in the activated-sludge plant during a period of from 12·3 to 16·4 hours. The impurities in the final effluent are shown in Curve D. Further information will be found in the Appendix (pp. 341–2)

Operation of the Sludge-Treatment Plant.

As the crude sludge is removed from the bottom of the primary sedimentation-tanks by pumping, and is not exposed to the atmosphere, there is an absence of aerial nuisance, and a considerable saving in labour. The water-content of the sludge pumped from these tanks averages approximately 93·8 per cent. and the average content of organic matter in the solids is 75 per cent. In the secondary sedimentation-tank is settled not only the lighter crude sludge but also the surplus activated sludge used in the pre-aeration tanks. The average water-content of the sludge pumped from these tanks is approximately 97·3 per cent., and the average

[1] "Factors in the Design of Sewage Disposal Works." *The Surveyor*, vol. lxxxviii (1935), pp. 397 and 433.

Fig. 10.



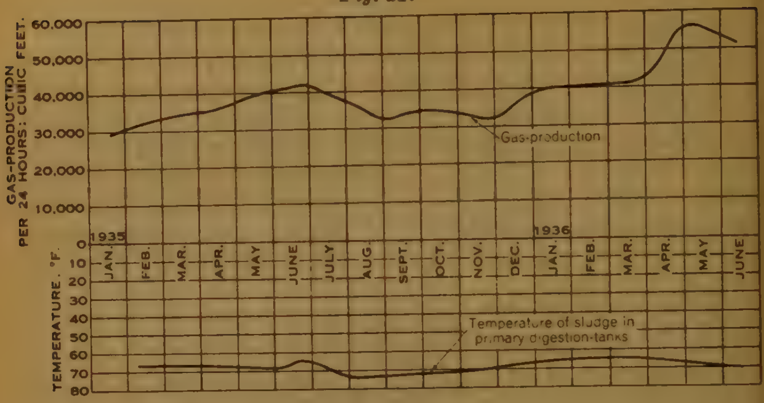
content of organic matter in the solids is 79·5 per cent. The corresponding figures for the digested sludge discharged to the drying beds are approximately 95·5 per cent. and 54·2 per cent. respectively. The variation in the temperature of the sludge in the primary digestion-tanks and in the rate of gas-production is shown by the curves in *Fig. 11*, p. 338.

Comparative Costs of Oxidizing Sewage.

In a previous Paper by the Author[1] comparative figures were

[1] "Some Methods and Costs of Oxidising Sewage." Proc. Association of Managers of Sewage Disposal Works, 1931, p. 100.

Fig. 11.



given of the cost of oxidizing sewage in five different plants in operation at the works of the Board. In none of the five cases was complete treatment given to the sewage by means of activated sludge. It is therefore of interest to compare the cost of oxidizing the sewage at the Board's Coleshill works with the costs previously given; this is done in Table II. Although the rate of interest on loans in connection with the Coleshill works did not exceed 4 per cent., it has been necessary, for the sake of comparing the operating costs, to take the same rate of interest as in the other cases, namely, $4\frac{3}{4}$ per cent.

Cost of Power.

Before estimating the net cost of the power to be obtained from the sludge-gas under conditions that have already been described, it is necessary to review certain factors:—

(1) To facilitate the control of the sludge-digestion process, and also to enable the capacity of the digestion-tanks to be very considerably curtailed, it had already been decided to heat the sludge in the primary tanks.

(2) The cheapest fuel obtainable for this purpose was sludge-gas.

(3) One-half only of the available gas would be sufficient for this purpose, and would be obtained by providing gas-collectors on half the primary digestion-tanks.

(4) Alternatively, the waste heat recoverable from engines consuming the whole of the sludge-gas would be sufficient for heating the digesting sludge. In this case gas-collectors would be provided on all the primary digestion-tanks, half of the collectors being regarded as chargeable to the heating of the sludge and half to the supply of power.

(Excluding the Cost of Land, Establishment-charges, Maintenance of Roads, etc. Rate of Interest on Loans taken as 4¾ per cent.)

Note.—Comparing the six plants generally, Nos. 1, 2, and 6 possess the advantage that the area occupied by works is reduced, as is also aerial nuisance and that due to the presence of flies.	Plant No. 1. Minworth works. Bio-flocculation Plant No. 1 and bacteria beds with fixed spray-jets.	Plant No. 2. Minworth works. Bio-flocculation Plant No. 2 and bacteria beds with rectangular travelling distributors.	Plant No. 3. Minworth works. Bacteria beds with fixed spray-jets.	Plant No. 4. Minworth works. Bacteria beds with rectangular travelling distributors.	Plant No. 5. Yardley works (formerly Coleshill works). Bacteria beds with rectangular travelling distributors.	Plant No. 6. Coleshill works. Activated-sludge plant.
Contributing population	235,000	375,000	470,000	125,000	100,000	67,000
Average flow treated: million gallons per 24 hours	7·5	12	15	4	3·18	2
Duration of treatment in bio-flocculation plant: hours	1	1	Nil	Nil	Nil	12
Average rate of distribution on bacteria beds:						
Gallons per acre per 24 hours	1,136,000	1,250,000	650,000	635,000	444,000	No bacteria beds.
Gallons per 24 hours per cubic yard of medium	117	111	67·5	56	48	
(Taking cost of bacteria beds with the necessary silt and humus tanks at the present price per acre of)	£15,000	£15,000	£15,000	£15,000	£15,000	
Total capital outlay	£144,130	£201,260	£345,000	£105,820	£114,856	£41,228
Per head of population	12*s.* 3*d.*	10*s.* 9*d.*	14*s.* 8½*d.*	16*s.* 11½*d.*	£1 3*s.* 0*d.*	12*s.* 3½*d.*
Per million gallons treated per 24 hours	£19,220	£16,780	£23,000	£26,455	£36,070	£20,614
Capital charges per annum	£9,447	£13,670	£21,700	£7,025	£7,464	£3,186
Repairs	£570	£1,040	£380	£475	£717	£280
Attendance	£1,310	£2,260	£2,875	£1,014	£806	£730
Electrical power, at 1·1*d.* per unit	£3,740	£3,613	Nil	Nil	Nil	£1,377
Total cost per annum	£15,067	£20,583	£24,955	£8,514	£8,987	£5,573
Annual cost per head of population	1*s.* 3½*d.*	1*s.* 1¼*d.*	1*s.* 0¾*d.*	1*s.* 4½*d.*	1*s.* 9½*d.*	1*s.* 8*d.*
Cost per million gallons treated	£5·51	£4·70	£4·56	£5·84	£7·75	£7·63
Oxygen absorbed in 4 hours: parts per 100,000 — Sedimented sewage	9·50	9·50	9·50	9·50	6·52	3·0
Oxygen absorbed in 4 hours: parts per 100,000 — Effluent	1·92	1·36	2·35	1·81	0·65	1·0

(5) From a population of 67,000 a reliable supply of 0·5 cubic foot of gas per head per day is obtainable under suitable conditions.

(6) Gas-engine-driven generating-sets of suitable size consume approximately 25 cubic feet of sludge-gas per electrical unit generated under full-load conditions.

(7) The number of electrical units which could therefore be generated by utilizing the sludge-gas amounted to 490,000 per annum, or sufficient to supply the requirements of the first purification-unit.

(8) It would be advisable to build a power-house large enough to accommodate additional engines in future, and also heating-plant.

(9) Only a portion of this building would be necessary to accommodate the engines required for a supply of power to the first unit of purification-plant.

(10) Taking the above circumstances into account, the proportion of the capital outlay chargeable to the power obtainable at present is taken as £7,570.

(11) Taking the rate of interest on capital expenditure as 4 per cent., the net annual cost of the power obtainable (490,000 units) is estimated as follows :—

	£
Capital charges	610
Attendance	630
Lubricating oil	135
Cooling water	25
Repairs	110
	£1,510

The cost per unit will thus be 0·74*d*.

As the requirements at the present time amount to approximately 400,000 units per annum, instead of 490,000 units as in the estimate, the actual cost per unit at the moment exceeds the above figure.

It will be noted that when the power-plant is increased to supply a second unit of purification-plant no additional charge for attendance will be incurred. It is estimated that the net cost of the power obtainable will then be 0·54*d*. per unit.

Cost of Gas.

The net cost of the gas obtainable may be estimated as follows :—

Capital outlay on five gas-collectors . . . . £3,300.

	£ per annum.
Capital charges	297
Painting	50
Repairs	20
Total annual charges	£367

Gas obtainable from a population of 67,000 at a yield of 0·5 cubic foot per head per day=12,200,000 cubic feet per annum.

Cost per 1,000 cubic feet=7·2 pence.

Taking the net calorific value of the sludge-gas at atmospheric pressure and a temperature of 60° F. as 675 B.Th.U. per cubic foot, cost per therm=1·07*d*.

Further information will be found in the Appendix.

Acknowledgement.

In conclusion, the Author wishes to express his thanks to the Engineer to the Board, Mr. H. C. Whitehead, M.Inst. C.E., for permission to present this Paper, and for the help and encouragement he gave during its preparation.

The Paper is accompanied by eleven tracings, from which the Figures in the text have been prepared, by three photographs, and by the following Appendix.

APPENDIX.

Later information is now available regarding the composition of the sewage and the degree of purification obtained at the Coleshill works.

Table III.—Average Results for 1936.

Sewage.	Parts per 100,000.								
	Suspended solids.	Free ammonia.	Albuminoid ammonia.	Chlorine.	Oxidized nitrogen.	Oxygen absorbed. Un-settled.	Oxygen absorbed. Settled.	Alkalinity.	Biological oxygen-demand.
Crude sewage	17·3	3·22	0·73	7·8	—	4·95	2·90	25·9	—
After sedimentation and pre-aeration	2·6	3·62	0·47	7·6	—	2·71	2·41	25·6	—
Effluent .	0·4	2·92	0·16	6·8	0·66	0·84	0·80	21·8	1·27

The operation of the plant has furnished detailed information regarding the quantity of sewage treated, the amount of compressed air used, the power-consumption, the quantity of sludge digested, and the amount of gas available. This information is set out in Table IV (p. 342).

TABLE IV.—COLESHILL WORKS (Estimated Population 60,000): SUMMARY OF RESULTS FOR THE YEAR ENDED 31ST MARCH, 1937.

	Rainfall: inches.	Sewage treated (not including sewage discharged over weirs of storm-water tanks): millions of gallons.					ACTIVATED-SLUDGE TREATMENT.					
							Air used: millions of cubic feet.			Activated sludge circulating: millions of gallons.		
		Total volume.	Volume in any one day.		Rate of flow per 24 hours.		Aeration tanks.	Pre-aeration tanks.	Total.	Aeration tanks.	Pre-aeration tanks.	Total.
			Maximum.	Minimum.	Maximum.	Minimum.						
	31·94	965·909	5·508	1·407	6·00	0·75	542·392	52·560	594·952	237·145	18·645	255·790
Daily average		2·646					1·486	0·144	1·630	0·650	0·050	0·700
Per million gallons of sewage treated							0·560	0·054	0·614	0·246	0·019	0·265

	Top-water- and sludge-pumps.	Pre-aeration compressors.	Lighting.	Auxiliary plant.	Unaccounted for.	Aeration-compressors, A.	Sludge-scrapers, B.	Activated-sludge pumps, C.	Total units used in the activated-sludge plant, A + B + C.	Total units generated.
Electrical units	17,928	34,917	36,073	18,980	181	267,798	6,205	32,632	306,635	414,714

HP. per million gallons of sewage treated per 24 hours in the activated-sludge plant . . . . . 17·75
HP. per million gallons of dry-weather flow per 24 hours . . . . . 23·48

SLUDGE-DIGESTION PLANT.

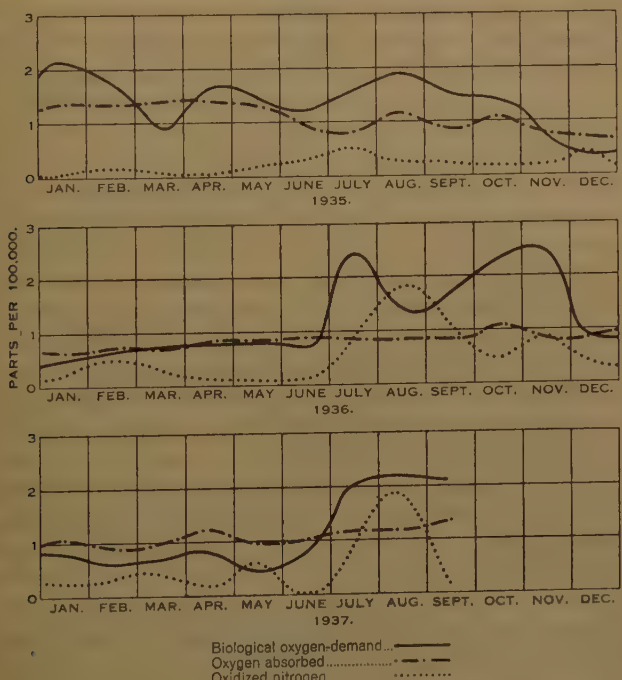
Gas consumed: cubic feet.		Gas to atmosphere: cubic feet.	Total gas: cubic feet.	Temperature of sludge in the primary digestion-tanks: °F.			Crude sludge to primary digestion.		Volatile matter in dry solids.	
Power.	Fires.			Average.	Maximum.	Minimum.	Quantity: cubic yards.	Water-content: per cent.	Crude sludge: per cent.	Secondary digested sludge: per cent.
12,488,608	392,600	4,220,373	17,101,581	69	74	64	47,223	95·63	74·7	51·2

Gas-yield per head per day . . . . . 0·78 cubic foot.

Discussion.

The AUTHOR exhibited a number of lantern-slides illustrating the works described in the Paper. The graphs in *Figs. 12* showed the biological oxygen-demand, the oxygen absorbed from potassium permanganate in 4 hours, and the oxidized-nitrogen content of the effluent discharged from the Coleshill works during a period of The Author.

Figs. 12.



2½ years. The increase in the oxidized nitrogen in the summer months and the accompanying increase in the biological oxygen-demand illustrated a statement recently made by Mr. W. T. Lockett [1] that under certain nitrification-conditions the 5-days biological

[1] W. T. Lockett, "A Contribution to the Literature relating to the Activated-Sludge Process," Public Works, Roads, and Transport Congress, London, 1937.

The Author. oxygen-demand figure alone was not a true guide to the real charact of the effluent. It would be noted that the oxygen absorbed in hours was not subject to much variation.

Mr. Frank. Mr. T. PEIRSON FRANK remarked that Birmingham was t largest town in England which was to be found towards the hea of a river, so that the local conditions would necessitate a very hi degree of purification of sewage. What was the minimum flow the river Tame ?

The Author stated that difficulty had been experienced on accou of the retention of sewage for some period in the large sewer, whi was 7½ miles in length. Had the local authority considered t advisability of adding chlorine to the upper part of that outfa sewer to delay the onset of septic conditions ?

On what grounds had the process selected for the purificati of the sewage been chosen in preference to other methods ? T works occupied a low-level site, and it had been desired to avo pumping ; were there any other reasons ? It was necessary to particularly guarded when trying to compare the costs of vario works, but in the last column of Table II (p. 339), the cost of t Coleshill works was shown as £7·63 per million gallons treated, an the purification, given by the test of oxygen absorbed in 4 hou was shown as 66 per cent. For the Yardley works the cost w £7·75 and the purification 90 per cent., whereas for the Minwor works, with bio-flocculation and bacteria beds, the cost was on £4·70 and the purification 86 per cent. Admittedly, the tan effluents varied in strength, and that might in some measure accou for those differences in cost, but it would be interesting to he the full reasons for the adoption of the particular process selected he did not imply that it was in any way unsuitable.

On p. 326 the Author referred to the use of surplus activated slud for treatment of the sewage before it was passed into the seconda sedimentation-tank. Mr. Frank had found that method useful a very similar type of operation.

Mr. Frank was interested in the reasons stated for adopting t type of costing given on p. 340. He was not an accountant, but imagined that there would be some accruing capital charges on t construction of further works when the plant was extended, whi would have to be added to the unit costs ; he was afraid that th would be fairly high, because in the Table on that page the capit charges for the power obtainable were given as £610 out of a tot of £1,510. On p. 341 the cost of the gas was mentioned. Wou there be any further charges for maintenance or attendance, su as those given on the preceding page ?

The Author and those associated with him were to be congratulat

n achieving a very neat layout of the works, readily capable of Mr. Frank. xpansion. The design had been based on a flow of 30 gallons per ead per day; was that regarded as ample?

He would like to know whether much difficulty had been xperienced in the construction of the centre well of the primary edimentation-tank shown in *Fig. 3* (p. 328). His own authority had ust completed the construction of some similar but larger tanks, nd that centre well had been the most difficult part of the work. 'he tanks in question were a little nearer to a river and in very ow-lying ground, and most of them had had to be constructed on iles. It appeared that the Coleshill tanks were founded on better round, as it was stated on p. 331 that the primary sedimentation-anks were constructed of concrete without reinforcement, and that he floors of the secondary sedimentation-tanks were of concrete inches thick without reinforcement. It would be interesting to now whether any cracks had appeared, or whether the ground was ood enough to prevent their formation.

The sewage at Coleshill might be classified as fairly weak, the uspended solids being 17·7 and the oxygen absorbed in 4 hours eing 4·56 parts per 100,000 by weight. At one of the works of Mr. Frank's authority the corresponding figures were 43·1 and ·82 respectively. The result given in the footnote to Table I p. 333) was, however, particularly good.

Referring to the operation of the oxidation-plant, it was men-ioned on p. 335 that a sample drawn from the aeration-tanks showed pproximately 10 per cent. of settled sludge after 1 hour. When n activated-sludge plant had been started at Barking about 6 ears ago, a discussion had taken place as to what percentage of ctivated sludge was to be put into the aeration-channel, and 10 per cent. was the figure that had been adopted for normal work-ing; the flow treated had since been increased from 10,000,000 gallons to 25,000,000 gallons per day, and was shortly to be raised to 60,000,000 gallons per day, and the figure had been temporarily reduced to 5 per cent. It was not intended to continue that, but the results were quite good, and still better ones had been obtained using 7½ per cent. of activated sludge. In the first place, however, it had been urged by some people that considerably more than 10 per cent. ought to be used.

From the curve in the lower part of *Fig. 11* (p. 338) it could pre-sumably be assumed that the maximum temperature in the primary digestion-tanks in the sludge-digestion process at Coleshill was about 73° F. It would be interesting to know whether the Author had tried working at a higher temperature. Mr. Frank had found that 90° F. gave the best liquefaction, and provision was now being

Mr. Frank. made to work tanks at 95° F. and even at 100° F. in case th should be desired.

Mr. Hetherington Mr. R. G. HETHERINGTON remarked that the Paper was of espec value in that it described not only the works but the results of th operation. Mr. Frank asked why the activated-sludge process h been adopted. In his own opinion one of the controlling fact was probably the fact that activated-sludge plant did not need considerable fall through the works.

On p. 324 reference was made to the tipping into the trunk sew of large quantities of cesspool-waste during the earlier period operation of the works. Presumably that did not now occur, b he would like to know whether it had caused much trouble, a whether any particular precautions had had to be taken.

He noticed that a varying water-level in the tanks was used equalize the flow at different times, and he would like to ask wheth the level was adjusted automatically or by hand.

On p. 326 the Author stated that surplus activated sludge w returned to the primary tanks : that was becoming almost standa practice. He went on, however, to refer to the fact that t secondary sedimentation-tanks had to be periodically emptied a cleaned to prevent fermenting sludge causing trouble. How oft was that found necessary ? On the same page it was mention that the decanted water from the primary sludge-digestion tan was passed through the secondary digestion-tanks. That w interesting, as it was not common practice, and he wondered wheth in avoiding one trouble by that means other troubles might m have been introduced. Having once separated that water from t sludge, it seemed strange to put it back again.

It was stated on p. 331 that the secondary sedimentation-tan had been constructed with holes in the bottom, to relieve possil upward water-pressure. That might be an excellent arrangeme in the case in question, but it was a system which the Minist of Health would view with some suspicion if it were widely use because sewage might escape through the holes. Presumab means were adopted to prevent that happening.

Mr. Frank had discussed the figures given in Table II, and so of them certainly seemed a little surprising. The cost per he for the Coleshill works was rather higher than for some of the oth plants, but it seemed to him that it was unfair to judge the wor too hardly on that point, as only one-fifth of the total works w yet in operation, and there were bound to be heavier expenses the first instalment. What he was puzzled about, however, w the figure of 3·0 parts per 100,000 given as the oxygen absorb by the sedimented sewage in 4 hours. That figure apparent

pplied to the sewage at a stage at which it had already received a Mr. Hetherington.
onsiderable amount of treatment, as surplus activated sludge was eturned to the sedimentation-tank, so that the difference between hat figure and the final one did not represent the full purification ffected. That, however, presumably did not apply to the figures or other works, so that the comparison did not give full credit to he Coleshill works. If, on the other hand, the figure in question eally applied to the strength of the sewage as received, the Coleshill orks had to deal with a very much weaker sewage than the others, nd that fact would have to be taken into consideration in comaring costs.

Mr. H. C. WHITEHEAD supplemented some of the information Mr. Whitehead.
iven in the Paper by data made available as a result of work arried out in recent months. A few months ago the dryeather flow at Coleshill had reached the designed capacity of the orks, and it became obvious that much of the plant would be apable of dealing with a greater load than that from a population f 67,000. Tests had accordingly been made, and it was now ossible to say that the sedimentation-tanks and aeration-plant ould be capable of giving satisfactory treatment to three times the ry-weather flow from a population of 100,000, upon the assumption hat there would be no increase in the strength of the sewage. The ortions of the works which would require enlargement before that opulation was reached were the separating-tanks, the sludgeigestion plant, and the storm-water tanks.

The exceptionally good results obtained in the sedimentationanks and aeration-plant were due, firstly, to the benefit derived rom a balancing capacity in the secondary sedimentation-tanks nd aeration-plant, and, secondly, to the improvement in the fficiency of the secondary sedimentation-tanks as a result of using he surplus activated sludge to improve flocculation in those tanks.

The Coleshill works illustrated how the inherent delicacy of the ctivated-sludge process might be minimized if care were taken to revent unnecessary loading entering the aeration-tanks. *Fig. 10*, . 337, well illustrated the extraordinary variation in load with hich the plant could deal. Incidentally, they were the first works mploying activated sludge as a means of complete purification in hich the whole of the power and heat required for the operation f the plant was provided by gas evolved from the bacterial digestion f the sludge.

Comparisons of cost were somewhat difficult to make, as the gures given in Table II, p. 339, related to the cost of oxidizing ewages ranging from the heavily-industrial sewage, with an oxygenbsorption figure of about 26 parts per 100,000, which was dis-

Mr. Whitehead. charged to the Minworth works, to the sewage of mainly domes[illegible] origin, and of only about one-quarter the strength, which was treat[illegible] at the Coleshill works. In such circumstances, of course, the co[illegible] were bound to be very much in favour of the works receiving t[illegible] strongest sewage. It should also be noted that the proved increa[illegible] in the capacity of the Coleshill works would bring about a co[illegible] siderable reduction of the costs shown in Table II.

Mr. Townend. Mr. C. B. TOWNEND observed that the Coleshill works might [illegible] regarded as a model plant, not only because it had been construct[illegible] to the most modern standards of design, but also because it w[illegible] being operated as a pilot plant to give an indication on a sufficien[illegible] large scale of what might be expected from the very much larg[illegible] works that was to follow. It was of especial interest in view of [illegible] similarity in many respects to the West Middlesex works at Mogden[illegible] although the latter was about twenty times as large. Both we[illegible] completely new works serving regional areas which had been recent[illegible] developed; their design had been unhampered by old plant [illegible] awkwardly-shaped sites; they had been constructed during simi[illegible] periods, and showed a marked resemblance in numerous points [illegible] design; in fact, in view of the radical advances over previo[illegible] practice incorporated in both works, it was remarkable that th[illegible] resemblance was not the result of any collusion between the tw[illegible] designing staffs. There were, nevertheless, marked points [illegible] difference in matters of detail. For instance, in the aeration-tan[illegible] at Mogden the sewage was made to travel a distance of 1,600 fe[illegible] in about 8 hours, every precaution being taken to prevent sho[illegible] circuiting. At Coleshill, on the other hand, the length of t[illegible] aeration-tank was 110 feet, which was traversed once only in abo[illegible] 14 hours, giving a rate of about 8 feet per hour as against 200 fe[illegible] per hour at Mogden. The Coleshill arrangement might possibly [illegible] productive of short-circuiting, which might explain fundamen[illegible] differences in operating experience.

Mr. Whitehead's remarks, giving the latest experience at Colesh[illegible] qualified to some extent what was stated in the Paper, and wou[illegible] probably also affect to some extent what Mr. Townend proposed [illegible] say. Whereas Coleshill employed a comparatively long aeratio[illegible] period of about 14 hours, with the very small air-supply of abo[illegible] 0·75 cubic foot per gallon of dry-weather flow, Mogden, with [illegible] stronger sewage, employed a normal detention-period of only abo[illegible] 8 hours but an air-supply about double that of Coleshill. The two plants, therefore, might be taken to be working at two extrem[illegible]

[1] D. M. Watson, "West Middlesex Main Drainage." Journal Inst. C.[illegible] vol. 5 (1936–37), p. 463. (April, 1937.)

the one with a long detention-period and low air-consumption, Mr. Townend.
e other with a minimum detention-period and high air-consumpon. Between those two extremes, it would be possible to have ıy number of combinations, and the best method of operation of ıy particular plant to give a specified result would depend on the lation between the cost of tank-construction and the price of ›wer. At Coleshill, as the ground was bad, construction-costs ere likely to be high, whilst the cost of power generated from udge-gas was low; it would therefore be expected that a low tention-period with a generous air-supply would have been lopted, particularly as it appeared that the utilization of sludgeıs had not been fully exploited. The 4,000,000 cubic feet of gas xhausted to the atmosphere during a year would have produced least 140,000 additional units of current, sufficient to raise the ›wer available to about 35 h.p. per million gallons per day of ry-weather flow, which was nearly equal to the figure for the ronger sewage at Mogden, where the short detention-period of hours was employed.

The economics of operating with the shorter detention-period ›uld be gauged from the figures given in Table II, p. 339. he cost of the Coleshill activated-sludge plant was given as £41,228, ith an annual capital charge of £3,186. At current rates of interest, ıat annual charge might be reduced to £2,400, but, even so, a ıving of over £1,000 per annum could be effected if the detentioneriod required were reduced from 14 to 8 hours. The extra current equired would be economically justified, even if it cost 1*d.* per unit, ut as at Coleshill it would appear that that additional power could e generated virtually at no cost from the gas at present wasted, he saving of £1,000 per annum would remain intact. Perhaps he Author would give some information on that point.

From the description of the early days of operation (p. 332), it ppeared that care had been necessary to avoid bulking due to ısufficient purification, on the one hand, and the rising of sludge ı the nitrification-stage, on the other hand. That might account or the frequent references in the Paper to the "sensitive activatedludge plant." Experience at Mogden indicated that when purifiation was taken to the point of advanced nitrification, the activatedludge process was remarkably robust in operation. It would be nteresting to know whether the design of the plant at Coleshill vould actually prevent purification being taken to such a point, ven if that were desired.

The Author had given in Table II some very interesting figures f the comparative costs of oxidizing sewage by different methods, ›ut in the complete absence of any qualifying remarks it would

Mr. Townend. appear that those figures should be studied with considerable reser No true comparisons of that kind were possible unless the plan considered were constructed on similar sites, served populatic of similar size, treated sewage of similar character, and produced similar standard of effluent. In the Table given, all those facto varied very widely. The capital cost of 12*s*. 3½*d*. per head giv for the activated-sludge plant of the Coleshill works was actua higher than both partial-treatment plants Nos. 1 and 2, and nea as high as the percolating-filter installation No. 3. That w unusual, and moreover the corresponding figure for the Mogd plant was only 6*s*. 6*d*. Presumably the price of electrical pow at 1·1*d*. per unit included transmission-charges and transform losses, but, even so, it was abnormally high. From all points view, Table II did not seem to do justice to the activated-slud process, either on grounds of cost or of oxidizing-capacity.

One of the most important innovations at Coleshill was the p aeration of the sewage by the surplus activated sludge, which wo otherwise be discharged to waste. The results were very sat factory, the reduction in the quantity of suspended matter passi on to the oxidation-plant being taken to a point never before reache That should lead to an increase in the purification-capacity of t plant as a whole, as Mr. Whitehead had already mentioned. Wh that pre-aeration was coupled with the ability to control and balan the rate of flow through the plant, the smoothing-out of the impurit load curve as indicated by *Fig. 10* was remarkable.

It was not always possible, of course, to employ balancing, wh required several feet of available head for its operation, but should be pointed out that in any ordinary works a similar effe was produced to a smaller degree by the lag occurring in the se mentation-tanks. Because of that lag, the strong daytime sewa did not reach the oxidation-plant until the late evening, when t rate of flow through the aeration-tanks was reduced by reason of t smaller flow of sewage then arriving at the works. In that mann the strong day sewage was given a detention-period longer than t average, whilst the weak night sewage arriving at the aeration-tan in the morning was pushed through at a much faster rate. Th effect was presumably made use of to some extent at Coleshi since the balancing capacity provided of 3·1 hours of dry-weath flow would not be sufficient in itself to give a constant flow to t aeration-tanks. At Coleshill, a capacity equivalent to 7 or 8 hou would probably be required for complete balancing.

Regarding sludge-digestion, the reduction in organic matter fro 75 per cent. to 51 per cent. was exceptionally good, although t gas-yield was not correspondingly high. It would be of intere

he Author could give the percentage grease-content of the sludge, Mr. Townend. ich might affect that yield very considerably. The high water-ntent of 95·5 per cent. of the digested sludge discharged to the ying-beds was typical where surplus activated sludge was dealt th. Perhaps the Author would say what had been the experience drying that sludge, to what depth it was pumped on the beds, at was the thickness of cake produced, what was the average riod of drying required, and whether any difficulties had been countered with the allowance provided of 1 square yard to every persons.

The cost of gas-collection was given as 7·2*d.* per thousand cubic t on the basis of 0·5 cubic foot per head per day. On the output tually available of 0·78 cubic foot per head, the cost would be *d.* per 1,000 cubic feet. Taking precisely similar items of cost, e figure for Mogden worked out at not much over 1*d.* per 1,000 bic feet, which could be regarded as negligible. That advantage, course, was due to large-scale operation and deeper tanks. The imated net cost of power of 0·54*d.* per unit would show a sub-ntial saving on outside sources of supply, particularly when nsmission- and transformer-losses were taken into account, ilst the absolute reliability of a self-contained plant was of vital portance when operation had to be maintained for 24 hours r day.

Mr. L. F. MOUNTFORT said that on p. 323 the Author stated: "It Mr. Mountfort. s estimated that the sewage would be chiefly of a domestic aracter, and would be readily capable of complete treatment by eans of activated sludge." Then, on p. 325, he stated: "In the sign of the plant provision has been made for the improvement the sewage to the utmost practicable extent before it reaches e oxidation-plant. In order that the sensitive activated sludge all not be adversely affected at times, precautions have been ken to shield the oxidation-plant . . ." from various effects. e activated-sludge process had now been in operation for about years, and he wondered whether it had still to be shielded and otected by every conceivable means from an occasional overload.

On p. 329 the Author referred to the use of rotary air-compressors the crescent type with sliding blades. The use of such com-essors was unusual, reciprocating compressors being generally ployed, and it would be of interest if the Author could give any formation regarding their probable maintenance-costs.

The figures given on p. 335 for the percentage of sludge in the ration-tanks were stated in the usual manner as percentage lumes after certain periods of settlement, but it was very useful be able to check the density of the sludge, and therefore it would

Mr. Mountfort. be of interest if the Author could state the corresponding pro- portions by weight.

With regard to the costs given in Table II (p. 339), he thought that the Author had been rather unfair to the Coleshill plant. In that Table the oxygen absorbed in 4 hours by the final effluent was given as 1·0 part per 100,000, whereas the figure given in the Appendix for a year's working was 0·8. Further, the oxygen absorption figure of 3·0 parts per 100,000, given for the sedimented sewage, probably applied to the secondary sedimentation-tank effluent. It was not clear whether the total cost of £41,228 included the pre-aerating equipment, but if that was the case the Author could fairly have shown a better result by stating the oxygen absorption figure for the primary sedimentation-tank effluent.

In *Fig. 8* (p. 335) the Author gave a curve which he labelled "Calculated time of retention of sewage in aeration-tanks (omitting consideration of returned activated sludge)." It was at present usual to divide the total tank capacity by the hourly dry-weather flow and to call the resulting figure the detention-time. Strictly speaking, however, it was not the true detention-time when there was a fairly large volume of returned sludge, and especially when, as in some cases, the capacity of a sludge-reconditioning tank was included along with the rest. In the case of a large sludge-return, a fairly large volume of the sewage had a very much shorter time, and smaller and smaller portions of the sewage had longer and longer times in the plant. He quite agreed that the arithmetical average of all those times would give the same figure as that derived in the Paper, but it was hardly fair to call it the detention-time, because if two plants had very different relative volumes of returned sludge but had the same detention-time as calculated in the Paper, they would not necessarily be expected to give similar results. The figure calculated was very useful, and it was necessary to have it in order to design the total tank-capacity, but it would be better to call it simply what in fact it was, namely, the total tank-capacity in terms of the hourly dry-weather flow.

Dr. Calvert. Dr. H. T. Calvert remarked that the Author had described a very interesting work of art of the Birmingham school. Works of that school were now numerous, and he was tempted to wonder whether they would remain as engineering monuments, in the same way that engineering monuments in waterworks practice had remained for nearly a century. Sewage-disposal practice had evolved at such a rate that sewage-disposal works had not in the past been so enduring as waterworks. The methods developed at Birmingham had been very successful; he had tried to discover the reasons for their success, and he thought the present Paper,

[ca]refully read, would be found to show all those reasons. It would [be] seen how the Author had marshalled the data for the design, [an]d how careful reasoning had been the guide in the choice of the [p]rocess adopted, so that the Coleshill works embodied all that was [b]est in modern sewage-disposal practice. He firmly believed that [th]e success of the Birmingham school was the result of very close [c]o-operation between the chemist and the engineer, and his friend, [th]e late Mr. F. R. O'Shaughnessy, had taken a leading part in [m]oulding the sewage-disposal practice that it had developed. Dr. Calvert.

With regard to the details of the Paper, he would have liked to [h]ear more about the use of marl, especially in regard to the quantities [e]mployed and the effects produced. On p. 336 it was stated that [th]e average content of organic matter in the sludge from the primary [se]dimentation-tanks was 75 per cent.; that seemed to him rather [h]igh, and it would be interesting to determine exactly what happened [t]o that organic matter in reducing its proportion from 75 per cent. [t]o 54·2 per cent. One of the most striking features of the Paper [w]as *Fig. 10* (p. 337), which showed how balancing of the flow [a]ffected strength at various stages of the purification-process.

In connexion with the data required for the design of sewage-[d]isposal works, he would like to remark that the Coleshill works [h]ad not been designed simply on the conventional figure of 30 gallons [p]er head per day to which Mr. Frank had referred.

Mr. W. T. Lockett observed that the Paper was of very great [in]terest to all bio-chemists, sewage-works managers and others who [w]ere concerned with bacterial processes in use for sewage-purification [a]nd for the disposal of sludge. The activated-sludge process for the [t]reatment of sewage and the successful production of effluents [c]ontaining nitrates, as operated at Coleshill, had engaged his own [a]ttention for many years; at the Middlesex County Council purifica-[t]ion-works at Mogden, where he now spent much of his time, sewage [w]as similarly treated, also with the successful production of effluents [c]ontaining nitrates. From the information given in the Paper, [h]owever, it would appear that the methods adopted at Coleshill to [o]btain nitrified effluents differed somewhat from those used at [M]ogden, the chief difference being in the method of operation of the [a]eration-unit. At Coleshill, a low air-supply was coupled with a [lo]ng retention-period; at Mogden, a relatively liberal air-supply was [c]oupled with a short retention-period. Comparing those methods, [h]is own experience led him to the conclusion that generally it was [b]etter to apply, within certain limits, relatively large volumes of [a]ir, provided the price of power was reasonably low, not only because [i]n the long run it was more economic to do so, but also because many [o]f the troubles that might occur with the activated-sludge process, Mr. Lockett.

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Mr. Lockett. such as high air-pressures, blockage of diffusers, and bulking sludge, were largely overcome and in many cases entirely obviat by a liberal air-supply. As an example, illustrating that poin the ridge-and-furrow type of diffused-air activated-sludge pla at the Withington works, Manchester, with which he had been co cerned, might be mentioned. Prior to 1921, although the purific tion-process could be successfully maintained for long periods wi a low air-supply and with a low air-consumption per gallon of sewa treated, yet bulking of sludge occasionally occurred, and on sever occasions the plant had to be placed out of commission for recond tioning of the activated sludge. With the adoption of a con paratively liberal air-supply along the lines suggested, and wi rather closer attention in order to prevent undue over-working, t effective capacity of the plant was increased by approximately per cent., bulking was eliminated, and the air-consumption in term of free air per gallon of sewage treated was actually reduce Operating the plant along those lines subsequently and up to 193 when he had left Manchester, sewage was continuously treated that unit, apart from a few days when certain plant repairs we being made, without loss of control of sludge, for almost 14 yea with an average retention-period in the aeration-chamber of approx mately 4 hours and an air-consumption of about 1·1 to 1·2 cu foot of free air per gallon of sewage treated. Another example w the Mogden plant, which, without any trouble of any kind, had be successfully operated with the production of highly-nitrified effluen since early in 1936.

The pre-aeration process described in the Paper was particular interesting and, as far as he was aware, Coleshill employed the fir pre-aeration plant using surplus activated sludge for colloi flocculation installed in England. At Mogden, certain laborato experiments along those lines had been carried out on the cru sewage, using 1 or 2 per cent. by volume of surplus activated slud and an aeration-period of ½ hour. With the stronger afterno sewages not much purification was effected; but with weak sewages the purification brought about by the same treatment w appreciable, and the treated liquors were generally much fresh than the original sewage. It was proposed to carry out furth investigations along those lines.

The Author mentioned that the activated sludge of the Colesh plant was low in density and tended to rise in the final separatin

[1] E. Ardern and W. T. Lockett, "Activated-Sludge Process: Withingt Works." Journal Soc. Chem. Ind., 1923, vol. xlii, p. 225T; W. T. Locke "Activated-Sludge Process: Aeration and Circulation." Proc. Ass Managers Sewage Disposal Works, 1924, pp. 92–110.

nk; and that, in order to weight the sludge and overcome that Mr. Lockett. fficulty, a suitable quantity of marl was added daily. Throughout s experience, Mr. Lockett had found that activated sludge of good ttling properties could be obtained by the use of a liberal air-ipply and a retention-period of the sludge and sewage in the aera-on-chamber long enough to ensure the presence of a substantial oportion of dissolved oxygen (about 50 per cent. of saturation), r at least some part of each day, in the mixture of sludge and wage passing from the aeration-unit to the final separating-tanks; , better, a retention-period suitable to obtain 1·0 to 2·0 parts of tric nitrogen per 100,000 in the final effluent.

To prevent rising of sludge in the final separating-tanks, reduction f the period of non-aeration of the sludge (namely, its detention in e final separating-tanks) to a minimum, and thorough conditioning r "activating" of the sludge in the mixed-liquor channels before assing the mixed liquor into the final separating-tanks, seemed to e effective; and the latter might be most satisfactorily brought bout by the use of aeration-tanks so designed as to minimize short-rcuiting as far as possible.[1] At Mogden, where the aeration-nits were obviously so designed that the possibility of short-ircuiting of fresh sewage-solids or partially-activated sludge was duced to a minimum, no practical difficulties had been experienced rough rising of sludge since the plant had been brought into opera-on in December, 1935.

The figures given in Table I (p. 333) were interesting, but for veral reasons the analytical figures given in Table III (p. 341) ere much to be preferred for ascertaining the performance of the oleshill plant, especially as they were average figures obtained y analysis of a number of samples taken over a long period. The alue of the data given in Table III, however, would have been uch greater if figures for the biochemical oxygen-demand of the ewage and of the influent to the activated-sludge plant had been cluded. Perhaps the Author could supply those figures.

With regard to sludge-digestion, the reduction in the organic atter of the crude sludge, as shown in Table IV (p. 342), from 74·7 er cent. to 51·2 per cent., represented a loss of 64·5 per cent. of the rganic matter, which was extremely good; but the gas-yield of ·78 cubic foot per head of population per day was rather low, and elow what might have been expected. At Mogden, with a loss of rganic matter by digestion of only about 46 per cent., the gas-yield er head per day was rather more than 1·0 cubic foot. Both the ercentage reduction in organic matter and the gas-yield obtained at

[1] *Ibid.*, p. 343.

Mr. Lockett. Mogden were similar to those obtained by digestion of sludge many other works. The volume of gas produced at Coleshill pound of organic matter digested was apparently only about o half of that usually obtained. Probably the Author could read explain those somewhat unusual results.

The Paper covered an exceptionally wide field and consequen it was only natural that it should leave a few loopholes for m criticism. None the less, one of its outstanding features, wort of much commendation, was its demonstration that by means of activated-sludge process a nitrified effluent could be regula produced on a large scale at a cost which was not prohibiti on that account, particularly, it was of considerable importance.

Mr. Cotterell. Mr. A. P. I. COTTERELL, referring to Table IV (p. 342), remark that nearly 25 per cent. of the gas produced by the digestion of slu was thrown away; though the actual quantity might seem relativ small, he would like to know why it was necessary to waste such large proportion of what was really a valuable product. Germ engineers would be horrified at the idea of losing over 4,000,0 cubic feet of gas every year. In Stuttgart, for instance, they h found it worth while to lay a pipeline several kilometres in length order that surplus sludge-gas might be taken into the general supp of gas for the city, and he could not help feeling that it was only within the role of engineers but was also their duty to that no avoidable waste occurred. There should be some good u for that gas, which would be worth a not inconsiderable sum p annum.

He had just returned from the United States of America, whe he had noticed that less importance was attached to the value of t gas for power-production, at any rate in such cities as Chicago a Milwaukee, than was the case in England; American authorit said, and perhaps rightly, that they would rather leave the nitr genous value in the sludge for treatment in other ways. The pr portion of organic matter removed from the sludge by digestion w mentioned on p. 337, and some part of that, he imagined, was nitr genous matter. Engineers in the United States seemed to developing fertilizing materials rather than using gas for pow and they were putting up very large works for that purpose. H own predilection was for the utilization of the gas produced fro sludge for power, lighting, and similar purposes, but it was unwi to overlook what was being done elsewhere.

Finally, he would like to say how valuable it was to have Pape such as that under discussion presented by those who were respo sible for the management and running of sewage-works after th were constructed.

Mr. W. H. Hillier remarked that the sedimentation-capacity Mr. Hillier. ovided was very large, being 16 hours dry-weather flow maximum, id the Author stated that that should ensure efficient separation the sludge. That should certainly be the case, provided that ere was no septic action in the tanks, but he thought that there as considerable danger of that occurring in hot weather, especially surplus activated sludge was present in the secondary sedimen-tion-tanks. It was common practice to provide both primary id secondary sedimentation-tanks with mechanical sludging devices that sludge-removal could proceed at frequent intervals while e tanks were in flow. He was surprised to find that such devices id not been provided in the secondary sedimentation-tanks at oleshill, and that the sludge was removed manually from those nks. He would like to know how often that had to be done to revent septic action developing.

The notes on the operation of the oxidation-plant were most teresting. It seemed to him that the rising of the sludge referred on p. 332 was due to the fact that the nitrification stage was not ery far advanced, relatively large quantities of nitrites but only nall quantities of nitrates being formed. When the period of eration was reduced, the oxidation was probably confined to the rbon-oxidation stage, nitrites were not being formed, and the ouble of rising sludge was eliminated ; but when the time of reten-on was reduced still further, bulking of the sludge occurred, which dicated that there was too little oxygen available to satisfy the xygen-demand.

Previous speakers had referred to the advantages of complete itrification ; it was certainly valuable where an effluent of a very high andard of purity had to be discharged, but where an effluent of not ich a high standard was required, owing to large dilution being fforded in the stream into which the effluent was discharged, it was referable to work only in the carbon-oxidation stage, with a smaller uantity of air per gallon of sewage and a correspondingly smaller ower-consumption, and to avoid the nitrification stage altogether. Ie had had experience of a plant worked in that manner, and none f the difficulties experienced by the Author had been encountered.

It was stated that the sludge was liable to sudden changes in haracter owing to changes in the composition or rate of flow of the ewage. That was a common experience, but the difficulty could e considerably reduced if a store of reconditioned sludge were vailable.

With regard to the addition of marl to weight the activated sludge, hat quantity of marl was added, and did that addition increase he quantity of sludge to be dealt with in the secondary sedimenta-

Mr. Hillier. tion-tanks ? If the bulk were not increased, the moisture-con of the sludge would presumably be decreased. Further, as activated sludge was repeatedly returning from the final tanks was repeatedly being dosed with marl, it might be expected t the marl would accumulate in the sludge-flocs and would there impair the flocculating qualities of the sludge. He was surpri that the organic-matter content of the dry secondary-tank slu was as high as 79·5 per cent., especially as the addition of m would tend to reduce the content of organic matter.

Mr. Wilson. Mr. M. F.-G. WILSON mentioned that on p. 330 the Author refer to the use of steel containing 0·35 to 0·5 per cent. of copper. committee of The Institution was studying the corrosion-resista of various alloy-steels, including those with 0·5 per cent. and per cent. of copper. That committee had found that the 0·5-p cent. steel was quite satisfactory, and that there was little or advantage in using the higher and more expensive percentage; would be interesting to know the Author's experience of it in co nexion with the gas-collectors.

The Author. The AUTHOR, in reply, observed that the activated-sludge proc had been selected for the treatment of the sewage at the Colesh works because, firstly, only a limited fall was available; second the process offered complete protection from smell nuisance; an thirdly, it was estimated that the sewage would be chiefly of domestic character, and readily capable of complete treatment means of activated sludge.

As Mr. Peirson Frank had pointed out, the local conditions neces tated a high degree of purification. The effluent was discharg into the river Tame at a point where the minimum flow was or 18 million gallons per 24 hours.

The allowance of 30 gallons of sewage per head per day had prov approximately correct in the case under discussion, where there w little or no trade waste and where the sewers were watertight. reply to Mr. Hetherington, so long as the tipping of large quantiti of cesspool-waste into the sewer had continued, chlorine had be added at intervals. During that period the activated-sludge pla had required all the protection that had been provided by pr aeration tanks and means of balancing the flow. For the latt purpose the extra capacity provided in the secondary sedimentatio tanks was equal to 3·1 hours' dry-weather flow and that in th aeration-tanks to 4·1 hours, a total of 7·2 hours' dry-weather flow The variation in water-level was controlled automatically after th penstocks in the effluent-weir wall of the aeration-tanks had bee set in a position determined by the method of trial and error. Th tipping of cesspool-waste had now diminished and, under the norm

onditions that now prevailed, the plant had proved robust. There vas no need to "nurse" it in any way. The Author.

In connexion with the proposed extensions to the Coleshill works, Ir. Whitehead had pointed out that, assuming that no increase took lace in the strength of the sewage, the plant could be made capable f dealing with a flow 50 per cent. greater than that for which it was esigned, without further expenditure on sedimentation- or aeration-anks. That would bring about a considerable reduction in the costs hown in Table II, p. 339.

Mr. Townend had made comparisons between the Coleshill works t Birmingham and the West Middlesex works at Mogden. The lans of the Coleshill works were approved by the Ministry of Health nd sanction to the loan received in December, 1931; the opening f the works took place in October, 1934. The opening of the Iogden works took place two years later, in October, 1936. The Author was glad to hear that the designers of the West Middlesex vorks had benefited from the pioneer works designed by the Drainage Board in Birmingham, where large-scale activated-sludge plants had een in operation since 1923.

So far as the Author was aware, there was nothing in the design of he Coleshill plant which, under otherwise suitable circumstances, vould prevent purification being taken to the point of advanced itrification, if desired. That had, however, been found unnecessary. The forward velocity of the sewage in the aeration-tanks was much ess than at Mogden. With that arrangement at Coleshill better lilution was given to the sewage as the result of meeting a well-erated liquid immediately on entering the plant.

With reference to the figures in Table II, the Author suggested hat engineers who were able to produce similar information should e encouraged to do so and not advised to wait until figures were vailable regarding plants constructed on similar sites, serving opulations of similar size, treating sewage of similar character, and roducing a similar standard of effluent.

In five out of the six cases given in the Table the effluent had an xygen-absorption figure of less than 2 parts per 100,000. The ercentage purification thus depended largely upon the strength of he sedimented sewage, and was of little value for the purpose of omparison. The capital outlay given excluded that on the pre-eration tanks and on the primary and secondary sedimentation-anks. The sedimented sewage referred to was the effluent from he secondary sedimentation-tanks, which had previously received reatment in the pre-aeration tanks. The surplus activated sludge was returned into the latter tanks and not into the primary sedimentation-tanks.

The Author.

It would be seen that, for the purpose of the comparison betw the six plants in Table II, the power for Plant No. 6 had been charg at the same price, namely, 1·1*d.* per unit, as for Plants Nos and 2 at Minworth, where the transmission charges were hea The actual cost of power at Coleshill was given on p. 340. In co sidering the capital outlay per head of population, due regard had be paid to the wide variation in size of the different works. T same remark applied to comparisons between the cost of gas Mogden and at Coleshill.

In connexion with the supply of power, the type of costing giv on p. 340 had been adopted for the information of those who wish to consider a plant for dealing with a certain quantity of sewage a who had no intention of doubling the plant at an early date. At Coleshill works provision had been made for increasing the pow plant at an early date. For the purpose of determining the cost power on p. 340, capital charges had been omitted on that portion the power-house which had been constructed for future use, which was not occupied at present. The Board would, of cour actually have to meet those charges.

Referring to Table IV (p. 342), it would be seen that the avera flow of activated sludge returned to the aeration-tanks was 32·5 p cent. of the dry-weather flow of 2,000,000 gallons per 24 hours. typical sample of the mixed liquor in the aeration-tanks show approximately 10 per cent. of settled sludge after 1 hour or 7 p cent. after 18 hours' settlement. 100 milli-litres of the latter settl sludge contained approximately 1 gram of dry solid matter. M Peirson Frank had mentioned figures relating to the activated-slud plant at Barking, which, apparently, worked under somewh similar conditions to those of the partial-treatment plants operat by the Birmingham Drainage Board at Minworth. In those plan the average flow of activated sludge returned to the aeration-tan was 12 per cent. of the dry-weather flow. A sample of the mix liquor in the flocculation-tanks showed approximately 2½ per cen of settled sludge after 1 hour and the same proportion after 18 hou settlement.

Dr. Calvert and Mr. Hillier had asked for further informati regarding the use of marl to cause the activated sludge to settle mo readily in the separating-tanks and, at the same time, to improve t effluent. The marl plant comprised two small tanks, each 6 fe by 3 feet in plan, in which from 3 to 6 cwt. of marl per day w mixed in water with the aid of an air-jet. A small stream of t mixture flowed continuously into the activated sludge returning the aeration-tanks. The marl did not impair the flocculating prope ties of the activated sludge, and the amount used was so small th

lid not appreciably affect the quantity of sludge to be dealt with The Author. the secondary sedimentation-tanks. Those tanks were emptied l cleaned out every 10 days. The time spent and the amount of rk to be performed in cleaning them did not justify the provision mechanical sludging devices under present conditions. The avier sludge deposited in the primary sedimentation-tanks was noved under water with the aid of mechanical scrapers.

With reference to the digestion of the sludge, the pre-aeration nks served a useful purpose in ensuring that the surplus activated dge was thoroughly incorporated with crude sludge before the o were pumped from the secondary sedimentation-tanks into the estion-tanks. Both primary and secondary digestion-tanks were the rectangular type, in which digestion proceeded in biological nes. By those means activated sludge was prevented from short-cuiting and finding its way to the drying-beds in a partially-gested state.

At Mogden a dry-weather flow of 40 gallons of sewage per head r day contained 29 parts of suspended solids per 100,000. At leshill a dry-weather flow of 30 gallons per head per day contained parts of suspended solids per 100,000. After allowing for the loidal matter in both sewages, and considering the actual gas-ld at Coleshill, a considerably higher gas-yield would be expected Mogden than the 1 cubic foot per head per day stated, assuming e sludge to be digested equally completely.

A typical sample of the sludge from the primary sedimentation-nks at Coleshill had a grease-content of 26 per cent. It had not en found necessary to raise the digesting sludge to as high a nperature as 90° F., and the Author was interested to hear of the ovision made by Mr. Frank for working at a temperature of 95° to 0° F. if desired. Water drawn from the primary digestion-tanks s passed through the secondary digestion-tanks so that as much ive material as possible might be removed before it was returned the sewer.

With reference to Dr. Calvert's question as to what happened to e organic matter in the crude sludge during the process of digestion, following figures might be of interest. In Table IV (p. 342) ures were given relating to the sludge and the gas evolved. ilizing those, it was found that the amount of volatile matter ich disappeared during digestion was 750 tons, and the weight of s produced was 484 tons, the difference being 266 tons. That ference included the organic nitrogen and carbon in the crude dge which were converted into carbonates of ammonia and nained in solution in the digested sludge liquor. Being volatile fore reaching 100° C., they were not measured as volatile matter

The Author. in the subsequent determination. Calculations based upon the amount of nitrogen in the crude sludge (4·98 per cent.) and in digested sludge (2·56 per cent.) showed that that difference accou for about 75 per cent. of the 266 tons. That was supported by figures for ammoniacal nitrogen in the digested sludge liquor.

With reference to the drying of the digested sludge, it was pur on to the drying-beds to a depth of 18 or 20 inches and mainta at that depth by refilling during the first three or four days. time required for drying varied between 6 and 10 months. average thickness of cake produced was 5 inches. Where circ stances permitted it was desirable to increase the area some above the allowance of 1 square yard to every 3½ persons.

The site of the works was composed of a fairly loose gravel lying marl. Very little bad ground had been encountered, alth some water had to be dealt with. No difficulty had been experie in constructing the deep sumps in the primary sedimentation-ta as they were in good stiff marl. In each secondary sedimenta tank there were holes in the floor, covered with a skin of concr they were provided to relieve any exceptional water-pres beneath the floor. Sewage could not escape, as cut-off walls ext ing down into the marl were provided beneath all walls of the ta The ground had been sufficiently good to prevent the formatio any serious cracks.

The air-compressors of the crescent type with sliding blades at the Coleshill works had proved as reliable as the reciproca compressors used under similar conditions at the Board's Minw works, and the maintenance-costs were approximately the same

Mr. Wilson had referred to the corrosion-resisting properti steel containing as much as 2 per cent. of copper. The steel use the gas-collectors at Coleshill, containing between 0·35 and 0·5 cent. of copper, had proved satisfactory. The Author had experience of steels containing the higher percentage.

In conclusion, the Author wished to thank those who had tributed to the discussion. He had tried to express his gratitud endeavouring to give adequate replies to the questions raised.

*** The Correspondence on the foregoing Paper will be publis in the Institution Journal for October, 1938.—Sec. Inst. C.E.

ORDINARY MEETING.

7 December, 1937.

SYDNEY BRYAN DONKIN, President, in the Chair.

The Scrutineers reported that the following had been duly elected as

Members.

William Eustace Maclean.
James Simpson Scotland Ramsay.
Robert Robertson.
John Symonds.
George Bate Edmund Truscott.
Stanley Walton-Brown, M.Sc. (*Durham*).

Associate Members.

Syed Mahmood Alam, B.Sc. (*Manchester.*)
James Alexander, B.Sc. (*Aberdeen*), Stud. Inst. C.E.
James Laurence Niall Alexander, B.Sc. (*Birmingham*).
Hakim Ali, B.Sc. (*Edin.*).
Walter Alston, B.Sc. (*Glas.*).
Thomas William Atherton, B.Sc. (Eng.) (*Lond.*).
Leonard Warner Bailey, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Arthur Gordon Baker, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Archibald Morton Ballantyne, B.Sc., Ph.D. (*Glas.*).
Maurice Frank Barbey, Stud. Inst. C.E.
George Smith Barrass, B.Sc. (Eng.) (*Lond.*).
Dhunjibhoy Cursetjee Rustomjee Baxter, B.Sc. (Eng.) (*Lond.*), B.E. (*Bombay*).
Albert William Beal, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
George Alexander James Begg, B.Sc. Tech. (*Manchester*).
William Osborne Belton, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Frederick Henry William Bevis, Stud. Inst C.E.
Geoffrey Morse Binnie, M.A. (*Cantab.*).
Paul Blackett Botcherby, B.Sc. (Eng.) (*Lond.*).
Gwynne Henry Brader, Stud. Inst. C.E.
Jack Erskine Bridger, Stud. Inst. C.E.
Robert Valentine Burns, Ph.D. (*Lond.*), B.Sc. (*Mass.*).
John Alexander Byles, Stud. Inst. C.E.
Robert Morrison Carter.
Cyril Gordon Chatham, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Charles William Clarke.
Stanley Gray Cockburn, B.Sc. (*Edin.*).
George Milner Cole, B.Sc. Tech. (*Manchester*), Stud. Inst. C.E.
William John Henry Cooke, B.A.I. (*Dubl.*).
George Henry Colin Cooper, Stud. Inst. C.E.
Brian Aldhous Copas, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Charles Howard Cousins, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Simon Cytryn.
José Francisco Da Silva.
Stephen William Dassenaike, B.A. (*Cantab.*), Stud. Inst. C.E.

John Freer Davies, B.E. (*New Zealand*).
Ralfe Davidson Davies, M.A., Ph.D. (*Cantab.*).
Vernon Corbett Davies, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Frederick William Dawkes, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Peter Arthur Vere Dolleymore, B.Sc. (Eng.) (*Lond.*).
Charles Philip Dreyfus, B.Sc. (*Manchester*).
James Maurice Dymond.
Wynne Samuel Edwards, Stud. Inst. C.E.
Arthur Sydney Elibank Erskine-Murray, B.Sc. (*Birmingham*), Stud. Inst. C.E.
Niazi David Fetto, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Alfred Charles Lindsay Fooks, B.E. (*New Zealand*), Stud. Inst. C.E.
Samuel Wakeley Beart Foss, B.Sc. (Eng.) (*Lond.*).
Sydney George Fox, B.Sc. (*Wales*), Stud. Inst. C.E.
Frederick George Frame, B.Sc. (*Leeds*), Stud. Inst. C.E.
Ralph Freeman, Jun., M.A. (*Oxon.*), Stud. Inst. C.E.
Joseph Richard Freer-Hewish.
Charles Wesley Gair, B.Sc. (Eng.) (*Lond.*).
Edmund Paley Starkie Gardner, B.Sc. (Eng.) (*Lond.*).
Roy Ellison Garrett, B.E. (*New Zealand*), Stud. Inst. C.E.
Eric Guy Brian Gledhill, B.Sc. (Eng.) (*Lond.*).
William James Glenn, B.A., B.A.I. (*Dubl.*), Stud. Inst. C.E.
Solomon Glover, B.Sc. (*Belfast*), Stud. Inst. C.E.
Hugh Quintin Golder, M.Eng. (*Liverpool*), Stud. Inst. C.E.
Andrew Francis Joseph Grant, B.Sc. (Eng.) (*Lond.*).
William Grant, B.Sc. (*Glas.*).
John Norton Griffiths, Stud. Inst. C.E.
Charles James Grimwood, B.Sc. (Eng.) (*Lond.*).
Ernest Wade Foxton Hall-Craggs, B.A. (*Cantab.*).
Arthur Salmond Hamilton, B.Sc. (Eng.) (*Lond.*).
Frederick Cyril Roberts Hands.
Wilfred Felton Harms, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Arnold Harris, B.Sc. (*Cape Town*), Stud. Inst. C.E.
William Gordon Harris, M.A. (*Cantab.*), Stud. Inst. C.E.
Hubert Sterland Haslam, B.Eng. (*Sheffield*), Stud. Inst. C.E.
Lyndhurst Gerald Delve Heal, Stud. Inst. C.E.
John Gratwicke Hamilton Heasman.
Ronald William Henderson.
Christopher Hinton, M.A. (*Cantab.*).
John Wright Hodgson, Stud. Inst. C.E.
Patrick Edward Michael Holmes, B.A. (*Cantab.*), Stud. Inst. C.E.
Philip William Hunt.
Eric Tom Hutt.
Howard Cecil Hutt, Stud. Inst. C.E.
Leonard Seddon Johnson, B.Sc. (*Birmingham*).
Humphrey Francis Brandram Jones.
Donovan Horace Leslie Keeble, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
William Kerr, Jun., B.Sc. (*Glas.*).
Robert Stanley Kibbler, B.Sc. (Eng.) (*Lond.*).
William Kilvington, Stud. Inst. C.E.
Paul Emile Klarer, Stud. Inst. C.E.
Frederic Martin Knight.
Christopher William Lacey.
Charles Kenneth Larard, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
David Lawson, Stud. Inst. C.E.
Hugh McDowall Lawson, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
John James Lee, Stud. Inst. C.E.
Alexander Leslie, M.A. (*Cantab.*), Stud. Inst. C.E.
Clarence William Little.
Philip Edward Luke, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.

Ivor Arthur Edmund Lyons, Stud. Inst. C.E.
Donald Graham McGarey, B.Sc. (Eng.) (*Lond.*).
John Ross McGibbon, B.Sc. (*Glas.*), Stud. Inst. C.E.
Duncan Alexander Whyte McMillan, B.Sc. (*Cape Town*), Stud. Inst. C.E.
Thomas McMillan, Stud. Inst. C.E.
Alexander Roderick Macrae, B.Sc. (*Glas.*), Stud. Inst. C.E.
George Alexander McWilliam, B.Sc. (*Edin.*), Stud. Inst. C.E.
Charles Denoon Margach, Stud. Inst. C.E.
Francis George Spence Martin.
Edward Stanley Mason, Stud. Inst. C.E.
Leslie Frank Masters, Stud. Inst. C.E.
Edmond James Moncrieff Matheson, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
James Louis Matheson, M.Sc. (*Manchester*), Stud. Inst. C.E.
Cyril Harry Hallahan Mercer, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Wilfred Merchant, B.A. (*Oxon.*), Stud. Inst. C.E.
Herbert John Miller.
Walter John Millns, Stud. Inst. C.E.
John Moore, Stud. Inst. C.E.
John Raymond Muff, Stud. Inst. C.E.
Hugh William Clarke McDermot Murray, Stud. Inst. C.E.
Neville Gresham Fountain Nash, B.Sc. (*Birmingham*), Stud. Inst. C.E.
Leslie Hamilton Newson-Smith.
Terence Andrew O'Neill, Stud. Inst. C.E.
Dryden Frank Osborn, B.Sc. (*Witwatersrand*), Stud. Inst. C.E.
Philip Luther Osborne, Stud. Inst. C.E.
Wilfrid Eden Parker, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Joseph Hyacinth Pelly, B.E. (*Sydney*).
Eugene de Chevalier Penso.
Eustace Oliver Edwin Pereira, B.A. (*Cantab.*).
Nicholas Achilles Photiades, B.Sc. (Eng.) (*Lond.*).
James Arthur Piachaud, B.Sc. (Eng.) (*Lond.*).
Hapugoda Rankotge Premaratna, B.Sc. (*Edin.*).
John Rosslyn Rhedynog Price, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Patrick Robert Reid, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Stig Gunnar Rikof, B.Sc. (Eng.) (*Lond.*).
James Samuel Roberts, Stud. Inst. C.E.
Wilfred Kirby Rodwell.
Wilmot Jack Roe, B.Eng. (*Liverpool*).
Andrew Cairns Ross, B.Sc. (*St. Andrews*), Stud. Inst. C.E.
Norman Andrew Forster Rowntree, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Eric Lucas Rumsey, Stud. Inst. C.E.
Eugene Sandler, B.Sc. (*Cape Town*), Stud. Inst. C.E.
George Ernest Sewell, Stud. Inst. C.E.
Isaac Park Simpson, B.Eng. (*Liverpool*), Stud. Inst. C.E.
James Keay Simpson.
Tambyahpillai Sivaprakasapillai, B.Sc. (Eng.) (*Lond.*).
Matthew Clarence Sloane, B.Sc. (*Belfast*), Stud. Inst. C.E.
Harold Edward Stopher, M.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
George Martin Swan, B.Sc. (Eng.) (*Lond.*).
James John Taylor, Stud. Inst. C.E.
George Thompson, Stud. Inst. C.E.
George Edward Thompson.
Harry Thompson, Stud. Inst. C.E.
Douglas Clifford Treacher, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Albert Edward Valler, Stud. Inst. C.E.
Reginald Charles Vaughan, B.Sc. (*Bristol*).

GEORGE WILLIAM DILLWYN VENABLES-LLEWELYN, Stud. Inst. C.E.
BERNARD JOHN VICKARS, B.Eng. (*Liverpool*).
JOHN ECKSTEIN VINER, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
ROBERT LIONEL WAKEFORD, Stud. Inst. C.E.
GEORGE WATERS, B.Sc. (Eng.) (*Lond.*).
JAMES COGHLAN WATSON.
WILLIAM COLVILLE WATT, B.Sc. (*St. Andrews*), Stud. Inst. C.E.
WILLIAM RONALD WAUGH, Stud. Inst. C.E.
RICHARD CECIL WHITEHEAD, B.Sc. (*Birmingham*), Stud. Inst. C.E.
CHARLES JAMES WILKINSON.
CLAYTON STURGESS WILLEY, Stud. Inst. C.E.
IAN WILSON, B.Sc. (*Glas.*), Stud. Inst. C.E.
JAMES ALAN WINTERBOTTOM, Stud. Inst. C.E.
WILLIAM EDWIN WOOD, Stud. Inst. C.E.
HAROLD CUTHBERT SEDGWICK WOOLCOTT, Stud. Inst. C.E.
JOHN CAPIE WYLIE, B.Sc. (*Edin.*), Stud. Inst. C.E.
HARRY CHARLES YOUNG, Stud. Inst. C.E.

Associates.

Prince ALPHONSE DE CHIMAY.
ROBERT VARLEY.

The following Lecture was then delivered, and the thanks of The Institution were accorded to the Lecturer.

"Air-Raids as they Affect the Work of the Civil Engineer."

By Colonel WILLIAM GARFORTH, D.S.O., M.C.
(late Royal Engineers).

AT the outset, I wish to make it clear that this lecture deals exclusively with the passive defence measures advocated by the Government (through the medium of the Air Raid Precautions Department) in the event of war, and endeavours to show in what respect these measures are of particular interest to engineers, and how engineers can materially assist in their carrying-out by the judicious design of structures.

Modern conceptions of the probable effect of air-raids vary from the views of alarmists to the views of those who, while admitting that air-raids are possible, consider that they could never happen in the British Isles. Extreme precautionary measures, of an unreasonable nature, would be necessary to meet the extreme alarmist views, whereas at the other end of the scale no action at all would be contemplated. The Government recommendations are intended to provide protection on a reasonable scale. The Air Raid Precautions Department has prepared various publications on the subject, which can be obtained from H.M. Stationery Office, Kingsway, London W.C.2. Handbook No. 5, which will deal with structural precautions, is now under preparation. It is suggested, however, that engineers would be interested in Handbook No. 1, "Personal Protection against Gas," and Handbook No. 6, "Air Raid Precautions in Factories and Business Premises," which are now available.

BOMBS AND THEIR EFFECTS.

In order to get a clear conception of the whole subject, it is necessary to study the types of bombs which may be used in aerial warfare and their effects.

Classes of Bombs.

The different types of bombs that may be used are :—

(*a*) High-explosive bombs.
(*b*) Incendiary bombs.
(*c*) Gas bombs.

The nature, general effects and object of each type are as follows :—

(*a*) *High-explosive bombs.*—These bombs carry a high-explosive charge, and are used to do damage to personnel and material.

This damage is caused by :—

(i) Fragmentation, which is the bursting of the case into small splinters which are projected in all directions with great velocity.

(ii) Blast, which is the sudden air-pressure wave set up by the force of the explosion.

They can be fitted with a sensitive fuse so that they burst on impact. In this case the fragmentation and blast effect are combined; the bomb usually has a relatively light case, and the proportion of weight taken up by the explosive content is about 25 per cent.

They can also be used for penetration, in which case a delayed-action fuse is employed. The case of the bomb is usually strengthened and is of the A.P. (armour-piercing) or S.A.P. (semi-armour-piercing) type. This strengthening of the case reduces the explosive content to about 10 per cent. The fragmentation-effect is reduced according to the depth of penetration, and the general effect of such a bomb exploding underground is similar to that of a mined charge.

The A.P. bomb and the S.A.P. bomb are specially designed for attacking very highly resistant targets, so that against ordinary structures the use of the lighter-case bomb may be expected, though the S.A.P. bomb may occasionally be used as well.

High-explosive bombs are made in weights ranging from about 20 lb. to 2,000 lb. and possibly more. Those over 1,000 lb. are only likely to be used against very special targets, and weights of the order of 250 to 500 lb. are probably the most suitable for use against ordinary buildings.

(*b*) *Incendiary bombs.*—These range in weight from 1 kilogram (2 lb. 3 oz.) up to about 60 lb. It is expected, however, that the light 1-kilogram bomb will be more freely used than those of greater weight. This bomb will penetrate any form of light roof.

The methods of protection against this bomb are :—

(i) The bomb can be prevented from entering the building by the provision of a reinforced concrete roof 4 to 6 inches thick.

(ii) Where the roof is vulnerable, the top parts of the building should be rendered as fireproof as possible, and recognized fire-fighting methods employed to deal with the bomb as soon as it has come to rest. This will probably be on the first floor that it encounters.

(*c*) *Gas bombs.*—Information about gas is to be found in Air Raid Precautions Handbook No. 1, previously mentioned, and as protection against gas can be provided by hasty expedients and by the use of respirators, the subject is out of place in a lecture on structural precautions In the design of new buildings, however, a good deal can be done to enable these expedients to be carried out easily and effectively.

The Fall of Bombs.

From the point of view of structural protection, the only important considerations in regard to the fall of a bomb are the angle and the velocity of its arrival.

The angle of arrival is dependent on the height and speed of the aircraft at the moment of release, and also to some extent on the ballistic design of the bomb. Assuming normal design, the angle of arrival of a bomb from an aircraft flying at 200 miles per hour varies from about 38 degrees from a height of 2,000 feet to about 17 degrees from a height of 12,000 feet, these angles being measured from the vertical. It is therefore evident that bombs may strike the sides of buildings.

The velocity of arrival of the bomb governs the impulsive force of the blow due to the bomb itself, and the penetration effect. It varies with the height of release and the ballistic design of the bomb, but, assuming good design, the weight of the bomb does not affect it materially. The rate of increase of velocity with height falls off rapidly above a height of 10,000 to 12,000 feet.

Effects of Bombs.

In considering structural measures of protection, the following primary effects of bombing may have to be dealt with :—

(*a*) Impact of bomb.
(*b*) Penetration of bomb.
(*c*) Explosive effect :—
  (i) on impact,
  (ii) in an enclosed space,
  (iii) after penetration into yielding material,
  (iv) in close contact.

(*d*) Fragmentation effect and penetration of splinters.

In some cases there may be a combination of two or more of these effects.

(*a*) *Blow of impact.*—Whether the bomb penetrates, breaks up or fails to detonate, the energy of the blow must be considered. On an

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unyielding target the intensity may be very considerable. In the case of a 500-lb. bomb dropped from 10,000 feet the energy is of the order of 5 million foot-lb. concentrated on the actual small area of impact. If the bomb strikes more yielding material composed of a mass of separate particles such as earth, sand, or shingle, this energy will be more gradually absorbed and dissipated by retardation during penetration. The energy will also be dispersed in all directions, radiating from the point of impact or pressure. A structure supporting a depth of such material will have the energy transmitted to it but in the form of a distributed rather than a concentrated load.

(*b*) *Penetration.*—This is largely dependent on the nature and material of the target. With solid resistant material, penetration may be effected by "punching" a hole, by disintegration of the material according to its nature, or by a combination of the two. With a sufficient thickness of the material to prevent complete penetration, disintegration of the surface, for varying depths, or deflexion of the material will occur; flakes or masses of the material may also be projected away from the reverse side of the structure.

(*c*) *Explosive Effect.*—

(i) *On impact.*—Apart from fragmentation, the primary effect is blast due to air-pressure produced by the explosion. This pressure is of a very high order in the immediate vicinity of the explosion, but falls off rapidly in unrestricted air-space. The time taken for the pressure to act and die away is approximately 1/1,000 second, the force reaching its maximum in 1/10,000 second, after which it falls off abruptly, and is zero by the end of the 1/1,000 second. The pressure, therefore, sets up momentary and violent impulsive forces against any material or structure in the vicinity. The blast effect is greater in an upward direction, owing to surface reflection and the concentration of effort in the direction of least resistance.

(ii) *In an enclosed space.*—In this case the blast or pressure-wave is more effective in destruction, and may be subject to some degree of definition. A guide to its effect may be obtained from the methods of employing concussion-charges for demolitions.

(iii) *After penetration into yielding material.*—The disruptive effect is similar to that of a mined charge, and calculations made on that basis will cover the worst effects that may be expected. It should be noted that the maximum and effective radii of rupture determined by the mining formulas in military textbooks are based on the

destruction of an underground timbered gallery; the effects should therefore be less destructive if concrete or other strongly-constructed underground structures were under consideration.

(iv) *In close contact.*—The explosive effect where actual contact takes place with structural material such as walls is a case for consideration, though the chance of this condition is small; a bomb detonating on a road, or other hard surface, may, however, produce practical contact, and the effect on the material must be taken into account.

(*d*) *Fragmentation.*—The distribution of fragments or splinters from a bomb detonating on the surface in the open is more or less regular radially, though some small increase in a forward direction may be expected.

The greatest concentration of fragments will be from the zone of the centre section of the bomb; towards the nose it is smaller, and still less towards the base or tail. Splinters will be thrown out radially in all directions, the highest density being from 15 degrees above to 15 degrees below a plane through the centre of the bomb at right angles to its longer axis. Owing to the angle of arrival of a bomb the maximum concentration of projected splinters will be slightly upwards in a forward direction, and the reverse towards the rear. Fragmentation will be reduced where penetration takes place. The splinters are given a very high velocity by the explosion, and, though in the immediate vicinity of the explosion the blast or pressure-wave precedes them, at a distance varying with the size of the bomb the fragments overtake and proceed in advance of the pressure-wave.

Protective Measures.

Degrees of Protection.

From the details already given it must be obvious that complete protection against aerial bombardment can only be obtained at considerable expense. For instance, to obtain complete protection against a 500-lb. S.A.P. bomb about 60 to 70 feet of earth, 12 to 15 feet of reinforced concrete, or some proportionate combination of the two is required above the space to be protected. Such protection, however, need not be general, as although the 500-lb. S.A.P. bomb may be met with here and there it will normally be employed only against special targets. It is also most important to remember that in the basement of a well-constructed multi-storey building of, say, five storeys or more good protection can be obtained against the

500-lb. bomb fitted with a direct-action fuse or a slightly-delayed-action fuse. Normally, it is recommended that protection should be provided against blast, splinters, gas, and the lightest incendiary bomb.

The Effect of Blast.

Blast-effect is one of the most difficult subjects of investigation, and further information, as stated before, is being collected. If blast-pressures were sustained as in the case of wind-pressure there are few walls in existence which could resist them, but, as previously stated, they are only momentary, and this makes all the difference. It is thought that the investigations now being made may tend to show that buildings of normally-strong construction will not be affected by the blast of bombs exploding not nearer than 50 feet away.

Protective Measures.

One of the most effective measures of defence is dispersion. This is not a subject of direct interest to the engineer, but it should be mentioned in order to complete the picture. The greater the number of individual targets which have to be attacked, the less is the relative effectiveness of each attacking aircraft. Also, the greater the relative areas of open spaces, the less is the chance of a direct hit. For Government factories it is recommended that only one-eighth of any particular area should be covered by buildings.

Concealment or camouflage is another means of defence, and a special experimental establishment is about to be set up under the Air Raid Precautions Department in order to study that question.

Materials.

A Table showing the thicknesses of various materials which will provide protection against splinters from high-explosive bombs bursting not nearer than 50 feet away will be found in the Air Raid Precautions Handbook No. 6. It may be mentioned here that 13½ inches of brickwork, 1 foot of reinforced concrete, 1½ inch of steel, or 2 feet 6 inches of earth or sand will resist penetration by splinters. The wall-thicknesses required to give protection against splinters will also provide reasonable protection against blast.

Reinforced concrete 4 to 6 inches thick in roofs will provide protection against the lightest incendiary bomb, and 15 inches of reinforced concrete against incendiary bombs up to 25 lb. in weight.

Windows.

A note on the protection of windows is given in the Appendix (p. 376).

Types of Shelters.

Protection of a very high order is available in some tunnels, mines, underground galleries and caves; such places ought to be used as shelters wherever it is possible and safe to do so, but there are not enough of them to provide more than a very small contribution to the protection of the whole population.

The normal forms of shelter accommodation may be classified as follows :—

(*a*) Refuge-rooms in which no structural alterations have been made, but which by the use of expedients will give protection against gas and possibly against splinters.

(*b*) Protected rooms, in the construction or of adaptation of which structural precautions have been taken in order to provide protection against the direct impact of incendiary bombs and against the effect of blast, splinters, and gas.

(*c*) Shelters, which are rooms or other accommodation, in or apart from buildings, specially constructed as shelters.

Choice of Sites for Shelters.

Suitable sites for buildings to contain shelter accommodation are those in wide streets or squares, or facing open spaces. Sites adjoining buildings which have a low standard of resistance or abnormal fire risks, or which are near areas of military importance, should be avoided.

The more substantial the protection afforded by the roofs, walls, and floors of a building, the greater is the degree of protection afforded by a shelter therein. Generally speaking, multi-storey steel-framed or reinforced-concrete buildings are the most suitable for containing shelter-accommodation. They should be of fire-resisting construction, preferably with solid concrete floors and roofs and with solid cross walls or partitions, and should have a fair proportion of solids to voids in the external walls.

The following features in multi-storey buildings are disadvantageous :—

(*a*) External walls with a large proportion of voids to solids.

(*b*) "One-cell" floors, or floors with no partitions between external walls.

(*c*) Timber floors, or roofs or floors of weak construction.

(*d*) Doubtful lateral strength.

(*e*) Roof-lights with top glazing.

(*f*) Heavy objects on upper floors.

(*g*) High chimneys and parapets or heavy architectural features above roof-level.

(*h*) Buildings in which the occupancy involves abnormal fire-risks.

Location of Shelter Accommodation in a Building.

Having selected a suitable building on a suitable site, the best location for shelter accommodation must be chosen. In general, below-ground accommodation is best, as it is likely to have a greater degree of lateral protection, but it should not be lower than the water-mains or sewers. If this is unavoidable, the accommodation should be constructed as a tank with entrances above flood-level.

Where the walls are of sufficient thickness to afford lateral protection and the windows are of normal size, and providing that effective gasproofing is possible, the ground floor of a building may be considered as suitable for the location of shelter accommodation, but the doors and windows should be protected by sandbag walling or earth between shuttering to prevent the ingress of splinters.

Shelter accommodation on upper floors should generally be not lower than the second floor, and should have a minimum cover of two floors and the roof of the building.

Shelter accommodation should not be adjacent to internal courts or light-wells. It should not be sited immediately below large water-storage tanks, safes or heavy machinery which might be dislodged by a bomb and crush the roof of the shelter.

General Methods of Shelter-Construction.

A consideration of the best methods of constructing shelters in new buildings will often help in selecting the most suitable places for shelter accommodation in existing buildings. The main points to be considered are :—

(*a*) Reinforced concrete is the most satisfactory material, and reinforced brickwork is a good substitute.

(*b*) The floor forming the roof of the shelter accommodation should be made strong enough to take the weight of debris which might fall on to it should the upper parts of the building be damaged and collapse.

(*c*) The walls and roof of the shelter accommodation should not be pierced by pipes or conduits; if this is unavoidable, the holes should be efficiently caulked by means of some elastic material such as bitumen. Facilities for shutting off all gas and water services by means of suitable stop-valves should be provided.

(*d*) Where shelter accommodation is provided on upper floors, the window-area should not be greater than is required to provide normal lighting for peace-time use. The sills should be kept as high as possible and the individual windows as small as possible.

Capacity of Shelter Accommodation.

As a general rule, shelters should not accommodate more than 50 persons, and the number of persons who can use a shelter of a given size which has been protected against gas can be calculated from the whole surface-area of the shelter, including walls, floor, and ceiling, by allowing 75 square feet per person. If it is required to accommodate more people in a shelter than can be allowed on that basis, mechanical ventilation must be provided, and in that case the floor-space allowance of 6 square feet per person is desirable, but it should never be allowed to fall below 3½ square feet per person. A gas-proof filter through which the air is drawn will be necessary.

General Notes on Shelters.

All shelters should have at least two entrances protected by air-locks, and, if below ground, should have an emergency exit to the open air. Any doors exposed to the open should be protected from splinters by some form of traverse constructed of splinter-proof material. Such doors should open outwards, and should be made of solid wood at least 2 inches thick in order to withstand blast.

Some independent form of lighting should be provided for use in the event of failure of the electric supply, and in the case of large shelters a petrol-engine plant which will also operate the ventilating system is recommended. Such a plant should be in a separate part of the shelter accommodation, but its controls should be within the shelter itself.

In the case of smaller shelters heating will not normally be provided; where necessary, it should be electrical. It is desirable, both for heating and lighting, to avoid methods which consume oxygen.

Sanitary arrangements must be provided, preferably in ventilated accommodation off the main shelter. Dry or chemical closets can be used when ordinary sanitary accommodation is not possible.

Various items of equipment are recommended for shelters, such as a storage-tank for drinking-water, fire-extinguishing appliances, an independent wireless set, materials for gas-proofing, and tools for use if entrances are blocked by debris.

Wing-Commander E. J. Hodsoll, the head of the Air Raid Precautions Department, hopes that application will be made to his Department if any information on the subject of air-raid precautions is required.

APPENDIX.

Windows in Air-Raid Refuges.

1. In an air raid, unprotected windows are subject to the effects of the various types of bombs in the following ways :—

(1) They cannot resist penetration by splinters from bombs.
(2) The glass may be broken by blast, in which case fragments of glass are projected dangerously into the building.
(3) If the glass has been broken there is nothing to prevent poison gas passing through the window-opening from the outside to the room within.

2. The only way to prevent splinters passing through a window opening is to block the opening with sandbag walls or earth between shuttering 2 feet 6 inches thick or to provide a steel shutter 1½ inch thick, or to use other material of sufficient thickness to resist the penetration of splinters. Brickbats (broken bricks) about 12 inches in thickness between shuttering afford good protection against splinters. One of the best ways of dealing with a window-opening is to remove the window-frame and to fill in the opening with brickwork in cement mortar, taking care that the outer surface of this brickwork is flush with the outer wall of the building. This is often a cheaper method than the other forms of construction mentioned above, and it possesses the additional advantage of taking less time to carry out.

Protection against the splintering of glass cannot be given by fixing steel or wooden shutters of reasonable thickness on the outside of the window-frame, owing to the fact that the pressure pulse due to blast is conveyed through such shutters to the air behind them which, being compressed, will shatter the glass.

3. Apart from the danger of flying splinters, the object of maintaining intact the glass or a substitute for glass in a window-opening is, besides keeping out the effects of weather, etc., to prevent poison-laden air entering the room or shelter.

4. Air-pressure or blast due to a high-explosive bomb bursting a few hundred feet away from an ordinary window has the effect of shattering the glass and projecting the broken pieces with considerable velocity, causing serious danger to people inside the room.

5. Complete protection of windows from the effect of blast and splinters at the above-mentioned distances from a bomb can be given by constructing on the outside sandbag walls or earth traverses, 2 feet 6 inches thick between shuttering, which must entirely cover the window-opening and which must touch the brickwork with an overlap of at least 12 inches all round.

6. If complete protection cannot be given, the following recommendations illustrate various ways of obtaining a substantial degree of protection.

(i) The ordinary glass panes can be replaced by panes of laminated glass, which in the case of small window panes, say 10 inches to 15 inches, should be about $\frac{3}{16}$-inch thick. Larger panes should be thicker. Laminated glass will stand up to blast pressure. It may crack, but it will not break, and owing to the fact that it consists of thin layers of glass with thin sheets of celluloid in between, it will remain gas-tight. It must be clearly understood that laminated glass is necessary and that toughened glass will not serve the purpose.

(ii) The glass can be replaced by a non-inflammable celluloid material, such as cellastoid, $\frac{1}{16}$-inch thick, reinforced by $\frac{1}{2}$-inch mesh wire netting fixed rigidly to the frame of the window and in contact with the sheet of celluloid.

(iii) Celluloid $\frac{1}{64}$-inch thick can be stuck on to the inner surface of the glass pane with a transparent cellulose varnish, and wire netting ($\frac{1}{2}$-inch mesh) fixed immediately behind the reinforced pane. Blast-pressure at the distance above referred to may crack such a window, but pieces of glass will be prevented from being projected into the room.

(iv) Ordinary glass panes can be replaced by glass which is internally reinforced by wire netting. The danger arising from the projected pieces of glass is considerably reduced.

(v) If ordinary commercial cellophane is gummed on the inside of an ordinary glass pane, it will not prevent the glass being broken, but it will reduce the risk of pieces being projected into the building, the reduction being dependent on the thickness of the cellophane employed. Commercial non-waterproof cellophane can be bought in rolls 3 yards long and 17 inches wide. Two thicknesses of this cellophane should be applied to the window panes. Although a cellulose varnish is the best adhesive, ordinary gum can be used to stick the material to the glass, but it should be examined from time to time and regummed when found necessary. The moisture-proof variety of transparent wrapping material, such as is used on cigarette packets, requires a cellulose varnish as an adhesive. If several thicknesses of cellophane can be used, so much the better.

(vi) If celluloid or cellophane is not available, linen fabric (say from pillow-slips) should be gummed on the inside of the ordinary glass panes. This will not prevent the glass being broken, but it will reduce the risk of pieces being projected into the building. Alternatively, mosquito-netting can be used for this purpose.

(vii) If none of the above materials is available, as a last resort thick paper can be used in the same way, but this is not very satisfactory and it possesses the disadvantage of being opaque.

(viii) If the window is not protected against splinters and blast as suggested in paragraph (2), above, some provision must be made to block up the window-opening in the event of the window panes being fractured or driven in, as poison-laden air must be kept out of the protected accommodation. For this purpose, it is recommended that a frame should be kept handy which will fit accurately on the inside of the window, the inner surface of this frame being pressed against the window-frame by means of thumbscrews fixed at suitable intervals. Between the movable frame and the window-frame a rubber or felt gasket strip would help to make the joint air-tight. In this movable frame are fixed two layers of blanket material reinforced with strong wire netting ($\frac{1}{2}$-inch mesh) on each side. If, therefore, the existing window is punctured, the blanket window could be quickly placed in position to keep out the gas.

The blanket material with the wire netting on each side of it should be fixed to the side of the frame which will be nearest the outside (i.e. nearest the bursting bomb). The ends of the blanket material should fall over the sides of the frame and be securely fixed to it. The rubber or felt gasket should be fixed round the frame on top of the wire netting and blanket material. The bolt-

holes in the movable frame should be a good deal larger than t bolts, and the thumbscrews on the bolts should only be pres home sufficiently tight to make the frame gas-tight. The objec these precautions is to avoid extreme rigidity in the frame it and to enable it to have a small degree of movement which will y to the effects of blast.

In cases where there are difficulties in providing the spare blan window, it is recommended that an old piece of carpet or blan should be kept handy and nailed to the inside of the window-fra in the event of the glass being broken.

(ix) It would obviously be a waste of money to employ laminated glass other glass substitutes in windows which would be much expo to possible damage by splinters as well as blast. For instance, i building facing an open square the windows facing the square mi easily be damaged by splinters from bombs bursting in the squa On the other hand, for windows in the sides and rear of the build bounded by streets, the risk of damage by splinters would not be great. In the latter three sides of the building therefore, it mi be worth while replacing the glass in the windows by laminated gl or other glass substitutes to prevent this glass being damaged l blast. The only way to ensure protection of the glass in wind facing the square would be to cover them completely with a sandb or similar wall.

(x) Questions are often asked regarding the efficiency of various propo forms of hinged shutters, which in their normal position keep t window opening gas-proof, but are intended to be blown open blast and then by means of their spring hinges to close and seal t window opening. Actually, the inertia which has to be overco before such a shutter can operate on its hinges does not allow the bl pressure to open it quickly enough to prevent its being smash

(xi) Horizontal pavement lights (glass prisms) may be expected to sta up to blast pressure provided they are very securely fixed in th framework.

The same remark applies to glass prisms fixed into stallbo lights, i.e. vertical lights just above the pavement in the verti walls of the building.

If pavement and stallboard lights are protected by, say, a cou of thicknesses of sandbags, such protection will assist in distributi the blast-pressure impulse.

Sir Clement Hindley.

Sir CLEMENT HINDLEY, Vice-President, in moving a vote of than to the Lecturer, observed that, although the problem was an appa ing one, he believed that protection was feasible. The human ra had always been able in time to devise means of defence for its against attack. Such precautions as those that the Lecturer h

escribed were sometimes deprecated as unnecessary in Great Britain. Sir Clement Hindley.
hat view, however, was dangerously short-sighted, and it was of ie greatest importance that the exploratory work of the Air Raid recautions Department should be prominently and continuously rought to the notice of the engineering profession and the general ublic. In The Institution there was a body of people with expert nowledge anxious and ready to help, and he felt sure that they ould be of assistance if they were told what was required. The roblem of protecting factories and buildings was a gigantic one, nd first-hand knowledge of what could be done would be useful for ngineers in their work of design and construction. He would iggest to the Air Raid Precautions Department that they should iake use of the accumulated knowledge available in The Institution.

Sir LEOPOLD SAVILE, Vice-President, in seconding the vote of Sir Leopold Savile.
ianks, remarked that in future wars the civilian population would e in the front line of defence, and that at last people were beginning) realize that. As long as the civilian population was not emoralized, a country could go on fighting.

The vote of thanks was carried by acclamation, and was acknowedged by the Lecturer.

Wing-Commander T. R. CAVE-BROWNE-CAVE explained that the Wing-Comm. Cave-Browne-Cave.
estruction caused by an explosion was due to the production of a irge quantity of gas at very high temperature which therefore xpanded to a great volume. In doing so, it acquired great velocity way from the origin, and therefore exerted an enormous intensity f pressure as a reaction on any adjacent solid.

The effect of the explosion on the surrounding air was to emit one r more concentric pressure-waves having crests of extremely high ressure. The impact and reflection of such a wave on a solid wall as known to give a pressure–time graph having an intensely high eak followed by a period of more moderate suction. The effect of he wave upon an elastic or yielding partition was to give to it a ertain amount of momentum per unit area, which could be expressed 1 the form $MV = \int Pdt$, where M denoted the mass of the partition, the velocity imparted to it, and P the pressure acting at time t. he kinetic energy produced in unit area of the partition would be $MV^2 = (\int Pdt)^2/2M$. The failure or survival of the panel depended pon whether the amount of kinetic energy could be absorbed without ailure. That energy might be absorbed either by deformation of he panel or by movement of the panel on its supports. It was, owever, interesting to realize that the energy to be absorbed as a esult of a given blast was not constant, but was inversely pro-

portional to the weight of the partition per square foot. If the pa did not fail and was highly elastic it might rebound in recovery a perhaps produce sufficient velocity in the reverse direction to ca failure in that way. A suction phase in the blast-wave mig synchronize with that rebound and thereby increase the chance secondary failure.

While it was probably true to say that except at a very sm distance from a bomb the effect of blast would be insufficient cause much structural damage, the foregoing consideration d indicate that partitions should be made so that they could yie or if necessary collapse without transmitting to the main framewo of the building sufficient force to do it serious damage. It also show that windows, being comparatively light, would acquire great kinet energy from a given amount of momentum and would have to yie through a comparatively great distance if they were to avo destruction. Blast was therefore of great importance in its effe upon windows and skylights. The breakage of glass greatly reduc protection against gas, the flying or falling glass fragments involv great risk of injury to personnel, and broken glass in the stree might well restrict the use of pneumatic-tired vehicles.

Members present then asked a number of questions regarding a raid precautions.

The Lecturer. The LECTURER, in replying to the questions, stated that th effect of a 500-lb. bomb on the service-mains and sewers beneath typical London street had been investigated by a full-scale test. had been found that all mains were shattered; cast-iron mains we breached locally only, but the joints of steel mains were damaged considerable distances from the point of impact. A sewer two bric thick was only cracked. The maintenance of fire-fighting servic required special consideration on account of possible interruption water-supplies. A special Air Raids Precautions Handbook w being prepared for gas undertakings.

Shelters in the grounds of factories might be utilized for oth purposes in peace-time, so long as they could be rapidly cleared wh required. Upper floors of buildings should be cleared of inflammab material so that incendiary bombs could be more easily dealt wit A layer of sand on such floors was valuable. The probable duratic of air raids could not be estimated; shelters should generally provic for about 4 to 6 hours' continuous occupation. It was not considere necessary for local authorities to provide public shelters for th majority of the population; good houses should be selected to prepared and allocated as public shelters.

ORDINARY MEETING.

14 December, 1937.

Mr. SYDNEY BRYAN DONKIN, President, in the Chair.

The PRESIDENT said that he was very pleased to announce that the Council had that day unanimously appointed Mr. Edmund Graham Clark, M.C., B.Sc., Assoc. M. Inst. C.E., as Secretary of The Institution. He would like to add that that result had come about after the examination of about 1,100 applications.

The announcement was received with acclamation.

Mr. E. GRAHAM CLARK thanked the members present for the very cordial way in which they had received the President's announcement. He was proud to have been appointed to succeed his former chief and old friend Dr. Jeffcott as Secretary of The Institution, and he would do his utmost to further the objects and aims of The Institution and to be of service to its members.

The Council reported that they had recently transferred to the class of

Members.

ALAN ATKINSON, M.Eng. (*Sheffield*).
HAROLD PERCY FORGE.
JAMES ROWLAND HILL.
LESLIE CHARLES KEMP, B.Sc. (Eng.) (*Lond.*).
HAROLD MIDGLEY, M.Sc. (Eng.) (*Lond.*), B.Sc. (*Durham*).
ROBERT RODGER.
ROGER HERBERT STEED.

And had admitted as

Students.

WILFRED ERNEST ALGIE, B.Sc. (*S. Africa*).
WILLIAM TAYLOR ALLEN.
JOHN ALSTON, B.Sc. (*St. Andrews*).
EDWIN ALEXANDER ANDREWS.
ROY BAKER.
HOWARD IVOR BARNARD.
ERIC JAMES BEARDSWORTH, B.Eng. (*Liverpool*).
CHARLES NORMAN BENNETT.
GANESH GOVIND BHIDE, B.E. (*Bombay*).
WILLIAM DONALD BIRKETT.
GEOFFREY BRAITHWAITE.
PETER WINSLOW BULL.
THOMAS KINGSLAND BURDESS.
NICHOLAS FREDERICK BURSTON.
ALONZO DONALD CALTHORPE.

Norman Dobson Carter, B.Eng. (*Liverpool*).
Kenneth Child.
Angus Webster Christie.
Selwyn Rodney Cobb.
Donald William Cracknell.
Ian Duncan Cunningham.
Frank William Dale.
Raymond Davies.
Ivan Charles Barron Dickinson.
Charles Frederick Farncombe.
Terence Fitzherbert Fenton.
Robert Hesledon Foster.
Peter Moritz Rudolf Fraenkel, B.Sc. (Eng.) (*Lond.*).
George Frolich.
David Alexander Allan Fulton.
Edward Gibbons.
Dennis Raymond Goatly.
Basil Waldon Goodman.
Thomas Raymond Gravenall.
Goriune Thaddeus Gregorian.
Henry Lancelot Gullidge.
Robert Kenneth Hadfield.
Ronald Charles Harrower.
Philip Hartley.
Charles Kenneth Haswell, B.Sc. (Eng.) (*Lond.*).
James Carswell Henderson, B.Sc. (*Glasgow*).
Cecil Edward Hooton, B.Sc. (*Birmingham*).
William Robert Cameron Houston.
James McChleary Johnston, B.Sc. (Eng.) (*Lond.*).
John Delwyn Jones.
Stanley Edwin Jones, B.A. (*Cantab.*).
William Kirby Laing, B.A. (*Cantab.*).
William Booth Longworth.
Dennis William Sykes Ludlow.
Ronald Hubert McKibbin.
Charles Henry Macklin.
Duncan Kenneth MacTurk.
William Stansfield Marshall.
Frank Bentley Mayo, B.Sc. Tech. (*Manchester*).
Harry Mills.
Robert John Mindham.
Stanley Osborne Morton, B.Sc. (*Belfast*).
Frank Newton.
John Palmer.
Peter Edward Parsons.
Harold George Proctor.
Walter Reginald Race.
Chelamkoor Subba Rao, B.A. (*Andhra*), B.E. (*Madras*).
James Ritchie Reid.
Alexander Robb, B.Sc. (*St. Andrews*).
Geoffrey Pendleton Roberts.
John William Roulstone, Jun.
Dimitri Ruperti.
Palahennedi Hewage Dienies Silva.
John Vivian Nicholls-Silverlock.
Bertie Ernest Stangroom.
Thomas Martin Steven, B.Sc. (*Edin.*).
Joseph Swindells.
Charles Bertie Symons.
Frank Dennis Thomas, B.Eng. (*Liverpool*).
Richard Oliver Ryall Toop.
Douglas Henry Topley.
John Leslie Unitt.
William Smith Urquhart.
Charles Blackburn Walker.
Edward Charles Warden.
Henry Frederick John Weaver.
Stuart Geoffrey Webster.
John Walter Thomas Whitfield.
Colin Drake Yarrow, B.A. (*Cantab.*).

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5150.

"The Reconstruction of Chelsea Bridge." †

By ERNEST JAMES BUCKTON, B.Sc. (Eng.), and HARRY JOHN FEREDAY, MM. Inst. C.E.

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INTRODUCTION.

THIS Paper deals with the replacement of a suspension-bridge typical of 80 years ago, with its piled foundations, fixed portal-braced towers, iron suspension-chains, massive shore-anchorages, restricted roadway at each tower, and cast-iron ornamentation (*Fig. 1*, facing p. 384), by an essentially modern structure of deeply-founded solid piers, hinged towers without portals, self-anchored cables, unobstructed roadway throughout, and no ornamentation (*Fig. 2*, facing p. 385).

† Correspondence on this Paper can be accepted until the 15th April, 1938. —SEC. INST. C.E.

The old bridge was opened in 1858, as a toll bridge, but was entirely freed from toll 2 years after its acquisition in 1879 by the Metropolitan Board of Works. It was taken over by the London County Council when that body was constituted in 1888. Several years after its construction, it had to be strengthened by the addition of a third main chain on each side. Subsequently a 5-ton limit to vehicles using the bridge had to be imposed. The old roadway was 29 feet 4 inches wide, but was reduced at the towers to 22 feet 5 inches, and proved too narrow for increasing traffic requirements.

In 1926 the report of the Royal Commission on Cross River Traffic in London was issued; it recommended that Chelsea bridge should be rebuilt and widened to carry all classes of traffic.

Under instructions of the London County Council several schemes were prepared in Sir Frederick Palmer's time, the most favoured one being for a six-line roadway with a traffic-circus at the Chelsea end. Prolonged and unsuccessful negotiations took place for the acquisition of the necessary land, and in 1931 the question of reconstruction was, for economic reasons, temporarily shelved. Two years later it was decided to proceed with the scheme, adopting a four-line roadway, with traffic cuts at the Chelsea end instead of a circus, the Ministry of Transport agreeing to provide 60 per cent. of the cost from the Road Fund.

A road with a clear width to carry four lines of traffic on a bridge will efficiently serve considerably wider approach-roads which have to contend with obstructions, such as standing cars, inflow and outflow of traffic by side roads, pedestrian-crossings, trams, 'bus-stops, and so on, but when a bridge carriage-way has to be renewed it is usually more difficult to divert the traffic than in the case of a street with its branch roads. Also, roads are constantly being widened as property is rebuilt, and approach-roads always tend to be widened. For these reasons a bridge should be constructed, if possible, with a somewhat wider roadway than is necessary for the actual traffic-flow.

Type of Bridge Adopted.

Most of the bridges between Chelsea bridge and Tower bridge are rightly of arch type, in keeping with the London setting, but at Chelsea there is a straight reach of the Thames, with the Chelsea embankment on one side and the tree-clad bank formed by Battersea park on the other (Figs. 3, Plate 1). Chelsea bridge stands at one end of the reach and the Albert bridge at the other, and they are both rightly bridges of suspension type.

The setting is suitable for either type of bridge, but the number and size of spans of an arch bridge would have to be controlled by the

Fig. 1.

OLD CHELSEA BRIDGE: 26TH OCTOBER 1934.

5150.

Fig. 2.

NEW CHELSEA BRIDGE: 10TH AUGUST 1937.

adjacent four-span Victoria railway-bridge, whose openings had originally been planned to suit the old suspension-bridge. Apart from the objections to a four-span arch bridge, the "tunnel" effect on river-traffic of two arch bridges close together would have been most undesirable.

The former selection of a suspension-bridge was confirmed, and it was found best to construct the new piers on the site of the old ones, as it gave fairly satisfactory centre and shore spans and permitted both demolition and reconstruction of each pier to be carried out within the same cofferdams.

For a modern bridge the existing anchorages would have been inadequate, and their replacement by new independent anchorages in clay formation would have been expensive. It was therefore decided to adopt the self-anchored type of bridge, which has for some years past been developed abroad. In such bridges deep stiffening girders run from end to end, and the cables after passing over the towers are anchored to the ends of the stiffening girders, the ordinary anchorage pulls being replaced by thrusts in the stiffening girders, and the only shore attachment being by vertical links of minor importance. This type, in consequence of its heavier stiffening girders, has the additional advantage of greater stiffness and freedom from vibration.

When a six-line bridge was being considered it was found that the usual portal between each pair of towers would be most unpleasing in appearance. The height of the towers depends upon the span, which in this case was a fixed quantity. Consequently the height of the new towers had to be much the same as that of the old ones, but as the six-line road was to be three times as wide as the old road, the portal, which was rather attractive in the old bridge, would have been unsightly in the widened bridge; it was therefore decided to omit the portal and to support the towers laterally beneath the roadway. Later, when a four-line carriageway was decided upon, the same principle was adopted, although its need was less urgent.

The carriageway had to be connected to the existing roads on the Chelsea and Battersea sides at a level of approximately + 35·0 O.D., and had to rise to + 42·35 O.D. at the centre of the river to give the necessary headroom for river craft. It was decided to adopt a 1-in-45 grade for each shore span, joined by a vertical curve for the centre span of 7,031 feet radius under dead-load conditions (Figs. 4, Plate 1).

Theory of Self-Anchored Suspension-Bridge.

The self-anchored type of suspension-bridge adopted has been used in various places on the Continent and in America where site-conditions made it undesirable to subject the anchorages to horizontal

loads; a particularly fine example is the bridge across the Rhine at Mülheim near Cologne, with a main span of 1,033 feet and a roadway-width of 56 feet.

The main cables of bridges of this type are made fast to the ends of the stiffening girders, and the only load taken by each of the abutment-anchorages is the algebraic sum of the vertical components of the cable tension and the stiffening-girder compression, the end shear from the side spans, and the local loading of the portion of the road-way carried by the anchorage ends. The ordinary principles of suspension are unaffected. Stiffening girders, by their resistance to bending, distribute the effect of local live loads over all hangers of the main span or shore span, according to the position of the live load, and the hang of the cables automatically adjusts itself to such distributed additional loads, the towers pivoting and the suspension-cables over the unloaded spans adjusting themselves to the increased uniform tension in the main cables.

In the case of the new Chelsea bridge the lengths of the spans, as stated above, were fixed by existing river conditions. The height of the cables at the centre of the bridge was fixed at 13 feet above road-level to keep them above the line of sight, and the height of the towers was determined on the basis of economy combined with good appearance.

Loading.

Local Loading.

The loading adopted for the carriage-way of the bridge was based on the Ministry of Transport standard loading. For the floor-system, the conventional 20-ton tractor followed by three trailers, each weighing 13 tons, with an overall length of 75 feet was adopted as the live load, and the local loading for the footpaths was taken as 100 lb. per square foot.

Impact.

The impact-factor was varied by consent of the Ministry to make allowance for the maximum probable loading of portions of the structure with more than one line of traffic. Thus, for the buckle-plates, cross beams and stringers, the normal impact-allowance of 50 per cent. was used, but for the cross girders loaded with four lines of traffic it was reduced to 30 per cent.

Loading for the Main Suspension-System.

For the main suspension-system, a uniform load per square foot was taken, equal to 90 per cent. of the figure given in the Ministry of Transport equivalent-loading curve for the loaded length under

consideration, together with a knife-edge load of 2,430 lb. per foot width. These reductions were allowed by the Ministry of Transport in view of the probability that all four lines of traffic would not be fully loaded.

The footpath loading for the main suspension-system was taken as 84 lb. per square foot on an effective width of 12 feet for each footpath.

Lateral Loading.

For lateral loading, wind-effects were calculated on an assumed pressure of 20 lb. per square foot with full live loading, and of 50 lb. per square foot with dead load only, the wind-system being taken as a continuous girder over four lateral supports, one at each abutment and one at each pier. The wind-effects, however, were found to be small. As the main object of the lateral system was to combine the two stiffening girders into a wide compression-member to act as a thrust-girder to the suspension-cables, it was designed as lacing to a strut, and a lateral shear of $2\frac{1}{2}$ per cent. of the sum of the maximum compressive force in the two girders, due to the cables, was used in determining the sections of lateral members required, and also in designing the lateral-reaction details.

Temperature-Stresses.

As the bridge is an all-steel structure, equal changes in temperature of all the component parts produce no appreciable internal forces in the structure. The cables, however, are subject to direct radiation, and allowance was made for a relative difference in temperature between cables and stiffening girders of 20° F. The resulting moments in the girders were small, amounting to only 7 per cent. of the live-load moment.

Design of Piers.

It may be of interest to recall the types of pier-foundations adopted for various other bridges in London, and some of these are given below :—

Waterloo bridge (1817)	Piles and raft.
Old Southwark bridge (1819)	,, ,,
London bridge (1831)	,, ,,
Tower bridge (1894)	Caisson.
Vauxhall bridge (1906)	Timber cofferdam.
New Southwark bridge (1921)	Caisson.
Lambeth bridge (1932)	,,

Timber-piles-and-raft construction was undesirable but necessary in the days before caissons and steel sheet-piling were available.

In the case of Chelsea bridge the choice lay between caissons and open cofferdams, as the depth of the river and the level of the London clay would permit of either being adopted. Caissons give a dry bottom for founding, but the advantage of this can be over-rated and certainly cannot justify any appreciable extra expense for their adoption.

In this case open steel-sheet-piled cofferdams were adopted and proved quite satisfactory. Boreholes indicated that the surface-level of the London clay on the site of the piers ranged from — 16 to — 22 O.D. The proposed future depth of river dredging on this reach of the Thames is — 17·5 O.D., and it was decided to found the piers in London clay at — 40·0 O.D.

The piers were designed in mass concrete (Figs. 5, Plate 1) faced with granite above low-water level, and having voids in the upper portions to reduce weight. The calculated maximum gross pressure on the foundations was $4\frac{1}{2}$ tons per square foot at — 40·0 O.D. The original pressure at this level was about 2 tons per square foot, so that the net additional pressure imposed on the foundations was $2\frac{1}{2}$ tons per square foot.

Design of Abutments.

The new abutments are of only secondary importance, in that they have no great loading to sustain, but they had to be on the same site as the old ones, which were founded on timber piles, and it was not considered desirable to incorporate an old foundation of this type in the new structure. It was therefore decided to cut away the front of the old abutments as necessary, and to provide new granite-faced concrete abutment-walls (Figs. 6, Plate 1) with foundations taken down into the London clay at about — 19·0 O.D. These walls take the vertical anchorage-link loads and lateral end-reactions from the deck-system. Behind the new abutment-walls the old anchorages are cut away to form the abutment-chambers, in which the anchorage-ends of the stiffening girders are housed. The new granite-faced wing-walls are supported on concrete piles, which were more economical in the circumstances than deep foundations, and slip-joints are provided between them and the abutment-walls to mask any relative settlement that might occur.

General Articulation of the Bridge.

The stiffening girders are in three sections, there being a hinge near each tower. These stiffening girders are suspended by hangers from cables passing over and clamped to the top of each tower and

made fast to each end of the stiffening girders. Each tower is hinged at the base and is free to rock longitudinally, but is restrained transversely by side bearings, pin-hinged, on each side of the main rocker-bearing.

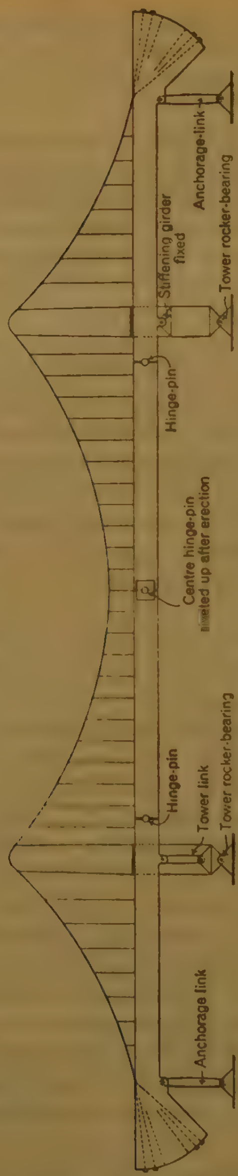
The towers are forked towards the base to stand astride the stiffening girders, thus providing for longitudinal movement between tower and girder. Some fixation is necessary, however, as without it the deck-system could be moved longitudinally, the towers adjusting themselves to such movement by inclining from their hinged bases, and unless otherwise held the whole structure would collapse soon after the towers had left the vertical. The necessary holding is effected at one of the piers by attaching the stiffening girders, by pin-jointed bearings, to the bases of the towers, so that relative movement is restricted to rocking, the girders being free to deflect. Additional vertical restraint or support is necessary to meet various conditions of loading, and each stiffening girder is further held by a link bearing at the other pier, and a link anchorage-bearing at each abutment. The bearings at piers and abutments prevent vertical movement of the deck-system at these points, and as three of them are link bearings the stiffening girders are left free to expand or contract longitudinally.

Lateral support to the deck-system is provided by side rubbing-plates on the tower-bases, in contact with short longitudinal wind-girders which span between the two cross girders nearest to the towers. Further lateral support is required at the flexible ends of the stiffening girders over each abutment, and this is provided by thrust-segments attached to each abutment and corresponding segments on the road-system. Movement is only restricted laterally.

The general arrangement of the articulation-system is shown diagrammatically in *Fig.* 7, p. 390. The system of hinge-pins used corresponds very closely to the normal methods adopted for spandrel-braced arches. The two permanent hinges close to the towers and the central temporary hinge simplify the erection. Under these conditions the system is statically determinate and the internal forces are computed from stress-diagrams.

To increase the local rigidity in the central span under live-load conditions, the stiffening girder above and below the hinge at the middle of the centre span was arranged so that at the conclusion of the erection of the structure and the placing of the surfacing of the carriageway and footpaths, joint-covers could be placed and the joint riveted to take the live-load moment and thrust. With this joint riveted the structure becomes singly indeterminate. Unlike the forces in the normal type of suspension-bridge, however, those in the self-anchored type can be calculated without taking cable-

Fig. 7.



GENERAL ARRANGEMENT OF THE ARTICULATION-SYSTEM.

deflexions into account. The vertical couple at any section is dependent only upon the horizontal thrust in the stiffening girder and the vertical distance between stiffening girder and cable, and this vertical distance is independent of all structural deflexions.

Use of High-Tensile Steel.

At the time of designing the bridge much progress was being made in developing high-tensile structural steels, and for the sake of economy it was decided to adopt high-tensile steel for the web and flange material of the deep stiffening girders. There was at that time no British Standard Specification to work to, and the subject had to be approached with caution, particularly in the case of high-tensile rivets. By close collaboration with the makers, satisfactory plates and rivets were obtained. Further particulars, with a summary of the numerous tests made, are given in Appendix II (p. 424).

Suspension-Cables.

Type of Cable.

For the main suspension-system of suspension-bridges several types of construction have been adopted in the past. In the structure of the original Chelsea bridge, and in the Menai bridge, wrought-iron eyebar-chains were used, and in several modern bridges heat-treated high-tensile steel chains of the same general type are employed. The Deutz bridge at Cologne is of this construction.

On grounds of economy and convenience of erection it was decided not to adopt this method of construction, but to use one of the types of wire-rope cable available. Four general types of wire-cable construction are in use.

In America, for long-span cable bridges, parallel-laid cables have been in general use for many years, the wire being spun in position from guide-wheels running on cat-walks and the cables afterwards compressed to a circular outline and protected by a continuous wire wrapping laid on by specially-designed machines. The wire, which is generally No. 6 gauge of 0·192 inch diameter and 100 tons per square inch minimum tensile strength, heavily galvanized, has been brought to a high standard of perfection from the experience gained in America on the many bridges constructed by this method. In a self-anchored structure with a comparatively short span this method of construction is practically very difficult and economically inadmissible.

A tendency is now apparent, at least for main spans up to 2,000 feet, for this method to be superseded by a construction using pre-stressed round strands bundled to a hexagonal outline, filled out with timber or aluminium packings to a circular shape, and wrapped as in the parallel-laid construction. The Isle of Orleans bridge, fully described by Mr. S. R. Banks, Assoc. M. Inst. C.E.,[1] is a fine

[1] "The Superstructure of the Island of Orleans Bridge, Quebec, Canada." Journal Inst. C.E., vol. 3 (1935–36), p. 357 (October, 1936).

example of this type. Here again, every effort is made to use the standard heavily-galvanized No. 6 gauge wire in the make-up of the rope. The interior of the rope-strand in this construction is not generally filled with any protective medium, owing to the rather open construction of the strand, and reliance is placed on the wire wrapping and its coatings of paint to protect the cable against corrosion.

In a short-span bridge with heavy loadings, the hanger-rods must be closely spaced and the wrapping of the cable (the portions adjacent to the hanger clips and those inside the girder must be wrapped by hand) becomes a matter of some difficulty. The method previously in general use for suspension-bridges of light construction erected in the British Empire consisted of various arrangements of stranded ropes. Such ropes, however, are not very efficient, and have large and uncertain permanent sets under load and a rather low modulus of elasticity.

For simplicity of erection and protection against corrosion, and in view of the specialized experience of British wire-rope manufacturers in the supply of locked-coil ropes, it was decided to adopt this type of cable, which had been used in the Cologne–Mülheim bridge and the new Alexander II bridge at Belgrade.

Construction of Cable.

The constructions of the cable and rope employed are shown on Figs. 8, Plate 1. Each cable is made up of thirty-seven locked-coil ropes bundled to form a hexagon. The rope construction comprises a Seale's lay (line-contact) core, one layer of radial wires and two layers of full-lock outer wires, and is specially designed to resist the compression across the rope occurring at the towers and at the turning-points in the anchorages. The working load on each rope is 35 tons. Protection against corrosion in individual ropes was obtained by passing each rope at every stage in its manufacture through a molten bath of bituminous material having the following properties :—

Melting-point (ball and ring test) . .	50°–60° C.
Penetration (100 grams for 5 seconds at 25° C.)	110–120.
Temperature required for use	110°–120° C.
Solubility in carbon disulphide . . .	99·8 per cent.
Mineral matter	0·4 per cent.
Viscosity at 200° F. (93° C.)	Material too thick to flow.
Viscosity at 300° F. (149° C.) . . .	220 Redwood seconds.

Further, each wire is galvanized. It was necessary to ensure a

fit between the component wires of the ropes in order to maintain a predetermined overall diameter so that the cable would fit the tower-caps, turning-point castings and hanger-clips. To this end the system known as " drawn " galvanizing was employed. After galvanizing, each wire was lightly drawn and the spelter thus brought down to a uniform thickness. After bundling, all joints in the outer surface of the cable were sealed with a plastic bitumen compound, and thereafter the entire outer surface painted with a coat of " Bitulax " semi-liquid bitumen compound. The aim throughout has been to afford as complete protection as possible against corrosion of the cable.

It may be noted that from comparisons with published results the elastic modulus of locked-wire constructions is not as high as that given by the type of round wire strand used in America.

The complete make-up of each rope is given in Table I, p. 394.

Calibration of Ropes.

The tendency to use single-strand ropes made up in hexagonal or similar shapes for the main cables of suspension-bridges, involving socketing of ropes to predetermined lengths before erection, has made it more or less standard practice to measure the ropes under a load corresponding to the average dead-load stress to which they will be subjected in the structure. The design of the pre-stressing plant involves the consideration of problems connected with the methods of measurement of load and length, the effects of temperature and friction, the effect of re-reeling on the non-elastic extension of the rope caused by the pre-stressing, and the handling of the ropes in such a way as to avoid damage to them.

In the case of the Cologne–Mülheim bridge the measurement of load was carried out by observations on pressure-gauges indicating the oil-pressure in the pulling-head cylinder. Frictional effects were eliminated by measuring the required rope-lengths while the tension in the rope was rising and again while it was falling, and by averaging the lengths thus obtained ; the assumption was made that the rising and falling frictions both in the pulling head and of the rope on the test-bed were equal. Owing to doubts whether the gauges were sufficiently accurate for this work, and to the difficulty in checking the assumption of equal frictions in the pulling head, it was considered desirable in the case of Chelsea bridge to adopt some further check on the pre-stressing loads, and a bar-dynamometer was used. The method of averaging measurements under rising and falling loads is also liable to some inaccuracy due to the hysteresis-

TABLE I.

CONSTRUCTIONAL DETAILS OF ROPE.

Diameter: inches.	Weight per 100 feet: lb.	Number of wires.	Diameter of wires: inches.	Range of stress of wire: tons per square inch.	Tension in wire: lb.	Mean tension: lb.	Aggregate tension: tons.	Torsion test on 8 inches: turns.	Number of dips in galvanizing.	Lays. Direction.	Lays. Length: inches.	Lays. Number per foot.
1·94 ± ½ per cent.	925·0 ± 2½ per cent.	1	0·188	30 (Mild Steel)	—	—	—	—	—	—	—	—
		6	0·170	85–95	4,327–4,837	4,582	12·273	10	Two of 1 minute and one of ½ minute			
		6	0·180	85–95	4,845–5,415	5,130	13·741	10		Left	7⅛	1·714
		6	0·137	85–95	2,805–3,135	2,970	7·955	14				
		17	0·160	Radial 85–95	4,760–5,320	5,040	38·250	6	One of 1 minute and one of ½ minute	Right	9⅞	1·215
		24	0·210	Lock 80–85	5,500–5,850	5,675	139·341	6		Left	13¾	0·815
		31	0·210	” ”				—		Right	17	0·695

Shaped wires.

	Equivalent diameter: inches.	Sectional area: square inches.
0·160-inch Radial .	0·1785	0·02502
0·210-inch Lock .	0·198	0·03079

Total aggregate tension . . .	211·560 tons
10 per cent. .	21·156 ”
Calculated actual tension . .	190·404 ”

effect in the extension-curves of wire ropes taken under these conditions.

Temperature-effects in the pre-stressing plant used for the Chelsea bridge ropes were comparatively slight, as the whole of the plant was under cover and protected from direct solar radiation, instead of being in the open air as was the case for the other bridges mentioned. Two methods of measurement are in general available. The measuring device may be allowed to expand or contract under variations of temperature, in which case the main object is to mark only at times when the temperature of the measuring device and the rope being pre-stressed are the same. Alternatively, the standard may be fixed by pedestals set in the ground, and corrections made for the difference between the temperature of the rope and the standard temperature. In the case of the Cologne–Mülheim bridge the former method was followed, the measuring device being a rope of the same make-up as the rope being pre-stressed ; the temperature-effects were thus automatically eliminated. For the Island of Orleans bridge the latter method was followed, but to eliminate doubt as to the rope-temperature the time of marking was restricted to periods when there was little variation of temperature, and all pre-stressing was carried out at night.

For Chelsea bridge, preliminary temperature-experiments were made at the site of the pre-stressing track in Birmingham, and these showed that there was always a period of at least 2 hours during the afternoon when the air- and rope-temperatures were sufficiently close to make rope-marking by the first method feasible. Accordingly, a method was developed, similar in outline to that used for the Cologne–Mülheim bridge, but using a thin tape for the measuring device. The periods during which the rope-marking was possible were restricted in the same way as in the second method because the thin tape followed the air-temperature closely, but ample time was available for the marking operations, as the method lent itself to the use of simple and expeditious marking devices.

Rope-Sockets.

For end-attachment the ropes are provided with cast-steel sockets. It was specified that the sockets should be of sufficient strength to break the rope without damage to themselves and without appreciable pulling-out from the socket.

The dimensions of the basket and the general details of socketing were submitted by the Sub-Contractor for wire rope. To prove the efficacy of his proposed design a test-piece of rope 12 feet long with a socket attached was tested to destruction, the slip of the rope

through the socket being carefully watched during the test. The first wire broke under a load of 182·7 tons when the actual slip amounted to only about 0·30 inch. This was considered satisfactory, and the socket shown in Figs. 9, Plate 1, was adopted.

The inside of the socket was left rough from the casting process, only sufficient fettling being done to remove the adherent casting sand, and a clearance was given at the bottom of the socket around the rope in order that the air might get away during the pouring of the white-metal filling. After the socket was threaded on to the rope the component wires were broomed out and thoroughly cleaned with solvent to remove the bituminous filling-compound. The wires of the outer layer were hooked back and the socket drawn on to a position determined from the pre-stressing operations. To facilitate the vitally-important pouring operation, the Sub-Contractors for the rope made arrangements whereby it was possible to pour the sockets of four ropes at the same time, and also to make necessary adjustments to the rope lengths immediately preceding this operation.

The sockets were pre-heated by blow-lamps, before pouring, to a temperature sufficient to ensure complete absence of moisture, and resin was placed in the broom as a flux. The white metal used was an alloy of 80 per cent. lead, 5 per cent. tin, and 15 per cent. antimony, with a melting-point of 450° F. Pouring was carried out at temperatures between 625° F. and 650° F. The whole operation was very carefully supervised.

To provide for small variations in length of individual ropes under erection-conditions, and to facilitate bundling of the cables, the sockets were made adjustable for length. The rope end of each socket was produced as a screwed skirt, to which was attached a collar arranged to butt against the seating-castings on the anchorage-end of the stiffening girder.

As the fanning of the ropes in elevation is not based on a single centre, but on a series of centres, and also as the ropes fan out in plan, the rope-axes are at varying angles to the circular end of the stiffening girder. To facilitate manufacture by standardization the abutting faces of the socket-sleeves and the seating-castings were made spherical and the sockets lined in during erection.

Stiffening Girders.

General Design.

The positions of the towers and anchorages for the bridge were fixed approximately by the positions of the piers and anchorages of the old structure. The proportions of centre span to side span thus

obtained were nearly two to one. In a self-anchored suspension-bridge of these proportions, with a symmetrical stiffening girder of constant depth, the maximum compressive stresses in the top flange of the centre span are greater than those in the bottom flange. In order to use the flange material to the greatest advantage, the centre-span top flange is made larger in area than the bottom flange. The dead-load thrust-line passing through the three hinge-pins, and parabolic in shape, is therefore eccentric to the gravity axis of the girder, thus producing moments under dead-load conditions. The degree of eccentricity is determined in such a way that the maximum compressive stresses due to combined live and dead loading in the top and bottom flanges are approximately equal.

In the side spans with a symmetrical girder the compressive fibre-stress in the top flange due to live loading of the near-side span is approximately equal to the compressive fibre-stress in the bottom flange of the same girder due to loading of the centre and far side spans. A symmetrical side-span girder was therefore adopted, but, in order to balance the moment caused by the overhanging ends of the anchorage portion of the girder, the point at which the cable is effectively anchored to the girder is lifted to such an extent as to provide, under average conditions, a balancing moment. The small changes in this moment due to varying cable-tension and anchorage-loading are easily provided for.

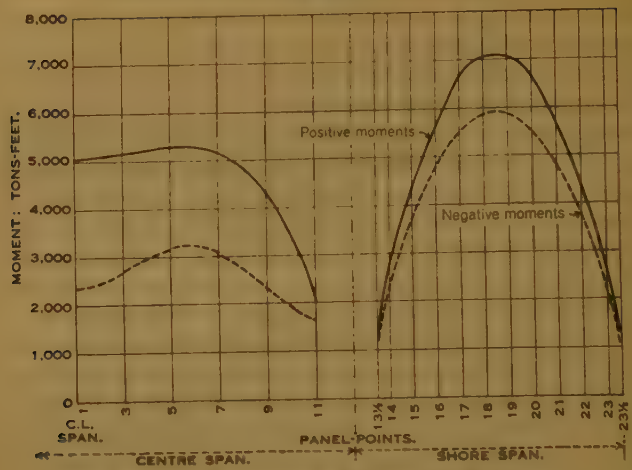
The stiffening girders in a self-anchored suspension-bridge perform the dual function of stiffening the roadway, thus lessening local cable-distortion, and of carrying the horizontal component of the cable-tension. In this case the structure is so shaped that under dead-load conditions no moment occurs in the side spans and only a small upward moment in the centre span. The maximum upward and downward moments at various points in the girder under live and dead load as given in *Fig. 10* (p. 398) are of interest. *Fig. 11* (p. 398) gives the distribution of stresses across the depth of the stiffening girder at the mid-point of the centre span under various conditions of loading, and may be taken as typical.

Each stiffening girder is made up of two plate-girders 8 feet 10 inches deep over angles, diaphragmed and battened together, generally as shown on Figs. 12, Plate 2.

Anchorage-Ends.

The general arrangement of the anchorage-ends of the girders is shown in Figs. 13, Plate 2. To maintain the line of the top flange of the stiffening girder for æsthetic reasons it was necessary to turn the cable down at the ends. This turning-point corresponds to

Fig. 10.

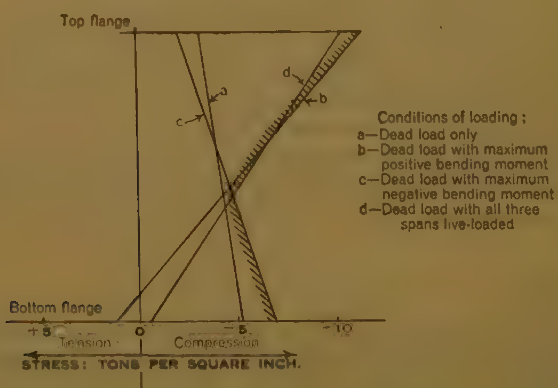


MAXIMUM MOMENTS IN STIFFENING GIRDER.

the cable-diversion bents frequently found at the abutments in normal-type suspension-bridges, and consists of a casting grooved to take the ropes, and allowing for their fanning-out into the anchorage proper (Figs. 14, Plate 2). The shape, size and angle of the casting are determined from a link polygon drawn for the forces in the ropes on each side of the casting.

The turning-point casting proper is mounted on rollers to allow for the extension of the end section of the cable, which is about 24 feet long. The rollers are supported by a casting riveted to the web-

Fig. 11.



STRESS-DISTRIBUTION ACROSS THE STIFFENING GIRDER AT THE MID-POINT OF THE CENTRE SPAN.

plates of the component girders, each of which is thickened by an outside ⅝-inch plate at this point to allow for the tensile stresses set up by the force from the turning-point casting (which amounts to 650 tons per stiffening girder) and to stiffen the plates against buckling.

The shape of the fan end is determined by the position of the top flange of the girder and by the space required for maintenance purposes between and on both sides of the fanned ropes.

Stiffening-Girder Supports.

In describing the general articulation of the bridge, reference has been made to the four bearings of the stiffening girders, one of these being fixed and the other three of hinged-link type to allow for movements due to stress and temperature-variation.

Each anchorage-link is pinned at the top between downward extensions of the anchorage side-plates suitably diaphragmed. At the bottom, it is pinned to a built-up structural-steel base connected to the anchorage-concrete by eight 2½-inch diameter bolts, each 27 feet long. The upper 8-foot lengths of these bolts were set in rain-water pipes, and were grouted-up after the bolts were tightened, in order to avoid as far as possible cracking the concrete near the surface.

The links are in two halves jointed together at the centre by a bolted spliced connexion and with 4 inches of packings of various thicknesses between the butting faces. This provision is made so that, if the main tower-foundations should settle, suitable adjustment could be made in the anchorage-links without unpinning the links.

The composition of the design load for the anchorage-links was as follows :—

Downwards.	Tons.	*Upwards.*	Tons.
Cable-tension effect (dead +live)	−108·0	Cable-tension effect (dead +live)	+152·6
End shear from shore span (live)	+132·0	Negative end shear from shore span (live) . . .	+128·5
Roadway over abutment (live)	+ 39·8	Local load acting as relief (dead)	−114·7
Local loads (dead) . .	+114·7		
	178·5		166·4

The stiffening-girder supports at the towers take the vertical loads due to main-span and side-span shear (either of which may be upwards or downwards, according to the position of the live load assumed), and the local dead load not carried by adjacent hangers. The

following are the design loads, the live-load effects being computed by influence-line diagrams :—

Downwards.	Tons.	*Upwards.*	Tons.
Live load	229	Live load	163
Local dead load . . .	76	Local dead relief, taken as	50
	305		113

The fixed bearing at each of the north towers is a built-up structural unit, and is shown in Figs. 15, Plate 2. Its lower face bears on the middle of the upper surface of the tower main bearing, and it is riveted on both sides to the tower-legs and bolted through the base. It is designed for the vertical loads given above, and for horizontal loads due to tower and anchorage-link inclination and longitudinal wind.

The expansion-bearings at the Battersea towers consist in effect of the lower half of the fixed-bearing structural detail connected by a double-pinned cast-steel link to the stiffening-girder web-plate projection. As in the case of the fixed bearing, the hinged connexion is necessary to provide for the upward reactions. Details of this bearing are also shown in Figs. 15, Plate 2.

Stiffening-Girder Hinges.

The three stiffening-girder hinges require to operate under two widely-differing conditions. During erection there is no thrust through the stiffening girders, and the hinges, particularly those near the towers, are subject to shear in a vertical direction. Although the centre support is arranged to bear evenly on both sides, the centre hinge is designed to take the full erection shear from one-half of the main span, to allow for possible uneven settlement of the piled temporary support or for partial failure of one-half of the mid-span erection jacking-system. All pins in these hinges are approximately 13 inches in diameter.

At the hinges adjacent to the towers, the sections of stiffening girder on both sides were pinned together in the works. It was thus possible to allow projections beyond the machined faces of the adjacent portions of the stiffening girders at the hinge. The detail therefore consists of half-pin bearing surfaces made up of the main webs and pin-plates totalling 9½ inches width per stiffening girder, and four fully-holed plates, two of which are connected to the outside of the girders and riveted to the main span girders, and the other two connected to the inside of the girders riveted to the tower-bearing

section of the side-span girders. Under erection conditions, the tendency of the pins to be forced out of their half-holes is resisted by a nearly horizontal component taken by the fully-holed plates.

The hinge-connexion at the middle of the main span, which is arranged to one side of the mid-span hanger-connexion in order to simplify the detailing, had to be made at site. To avoid danger of fouling during flotation of the third section of stiffening girder past the second section already erected, it was desirable to maintain a flush face to the abutting stiffening-girder ends in both sections. The horizontal component to maintain the pin in its half-seatings is therefore taken by a ring linking together two crescent-shaped forgings riveted on the outside of each component girder. To ensure that these forgings should be loaded horizontally, and hence to reduce their size, the forgings were first machined to suit the connecting ring and then the bearing surfaces ground away except for a width of 4 inches on the horizontal axis. Details of the centre hinge are given on Figs. 12, Plate 2.

To facilitate the insertion of the pins, each pin is chamfered at the entering end and the pin and hole diameter is made $\frac{1}{32}$ inch smaller for the inner component girder than for the outer component girder, the pins being inserted from the footway side in each case.

The hinge-pins at the tower hinges are kept in place by angles passing across the flush-machined outer faces and bolted to the main-span stiffening-girder end. Those at the centre hinge are kept in place by a bolt passing through a hole bored right through the pin and connecting together two circular locating plates bearing on the rings connecting together the two sections of main-span stiffening-girder. These bolts were made 12 inches longer than their final size to provide a ready means of drawing the pins into position at site, the projecting ends afterwards being cut off.

Towers.

The absence of portal-bracing is perhaps the most unusual feature of the structure (Figs. 16, Plate 2). The lateral stability of the towers depends therefore on the fixing of the tower-bases on the piers. Whilst for normal wind-pressure and eccentricity of loading it was found to be feasible to rely on the rigidity of the main tower-bases, it was considered desirable to provide for such a contingency as an aeroplane crashing into the towers by the use of side-support bases capable of taking tension.

The general make-up of the towers consists, near the base, of two plated-box columns passing on each side of the stiffening girder,

the flanges of which are cut back on both sides to suit. Above the top of the stiffening girder, the two box-columns are joined together by three plates, two on the outside face and one across the centre. The tower tapers $4\frac{3}{4}$ inches on each face from a point 6 feet above roadway-level to the underside of the tower-cap casting. For architectural reasons the joint-covers on the sections of the towers above road-level are placed inside the tower, and the outer plates therefore appear flush, except near the base where a plinth plate is used to give a finish, and at the corners where the plates are cut back from the heel of the corner angles to give a neat corner and vertical lines to the towers.

The two component box columns rest at the bottom on the main tower-bearings and are joined together at their bases. The towers are of the rocker type, and the bearing surfaces of the main tower-bearings meet in line contact, the pressure-concentration being backed up in both upper and lower bearings by a mass of metal.

Projecting up and down stream, side-support structural bases are built out resting on side-support bearings spaced at 18 feet between centres. The side-support bearings consist of upper and lower castings shaped in elevation to agree with the tower main bases and hinged together by 6-inch diameter pins. To resist upward thrust, the upper casting is riveted to plates which project downwards from the main structural-steel section and the lower casting is anchored into the pier by bolts.

The centre-line of the side-support bearing-pins coincides with the line contact of the main bearings when the tower is vertical; small forces of no importance arise when the tower is pulled back during erection and under live-load movements.

As it was impracticable to arrange the projecting side bases fully to stiffen the tower-legs, owing to the necessity for clearing the underside of the roadway and footways, there is a section of the tower-leg laterally unsupported. The loading of the main tower-bases by dead and live loads tends to produce secondary moments in these unsupported sections. To eliminate this as far as possible, each set of the side-bearing castings were wedged apart during erection to the limits of the clearances of the pins in the holes.

Expansion-Joints.

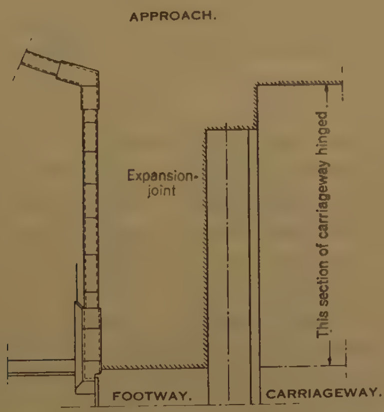
In order to provide a clean finish at the abutments, the expansion-joints are of a rather complicated layout (*Fig. 17*). Provision for longitudinal expansion had also to be considered in conjunction with the effect of the side-span vertical deflexion on the overhanging end

of the stiffening-girder anchorage, which is carried on about 24 feet beyond the anchorage-link.

The end section of the carriageway between the last cross girder and the abutment-wall is carried on rocking plates at both ends, allowing a slight variation in the relative levels at the two ends of the section. Above the joint at the last cross girder the roadway-supporting steel is broken and a ½-inch bitumen joint provided throughout the roadway and curb concrete at the granite curbs.

The abutment-walls form a large chamber in which the anchorage-ends of the stiffening girders are housed, and the walls project upwards on the return faces to form the parapets of the footways. It was desirable that the footways over the abutments should be

Fig. 17.



LAYOUT OF EXPANSION-JOINT AT ABUTMENT.

fixed, not to the bridge but to the abutments, and the lateral expansion-joint for the footways is therefore over the river-wall of the abutment. No special provision was necessary at this joint for vertical movement, as the joint nearly coincides in position with the anchorage-link. The carriageway and footway expansion-joints had therefore to be connected by a longitudinal joint. On the footway this runs under the edge of the screen-plate and is hidden. It is then returned across the end of the stiffening girder by a finger-type cast-iron joint to allow for the longitudinal expansion and the small vertical movement at that point. The connexion is completed by a ½-inch-wide joint between the curb vertical plate of the roadway and the ballast-angle of the footpath. The joint is extended along the lower edge of the stiffening-girder sloping plate to the lateral expansion-joint in the carriageway over the last cross girder.

Lateral expansion-joints in the roadway-surface consist of chequered plates, and the greatest exposed width of steel is about 2 feet.

The Post Office cables are provided with 8-foot-diameter expansion-loops under the footway bounded by the abutment-walls, and a steel platform is arranged to give support and to provide easy access.

Floor-System.

Lateral Bracing.

The lateral system consists of K-type bracing attached to the underside of the cross girders, each bay of bracing normally spanning two panels of the floor-system as shown on the key diagram in Figs. 18, Plate 3.

To accommodate the contraction of the stiffening girder under load, expansion-castings of 35/40-ton steel are introduced at the points marked X on the key diagram. Details of these castings and their connexions are given on Figs. 18, Plate 3. At the point marked M on the key diagram, where the abutment expansion-joints are situated, the arrangement is such as to allow movement in both vertical and longitudinal planes but to resist any tendency to move in a lateral plane. A hinge-casting is bolted to the underside of the cross girder and is connected to a yoke-casting by means of a 4-inch-diameter pin. The yoke-casting is furnished at its lower ends with $6\frac{1}{2}$-inch diameter bearing bosses fitting into suitably-holed plates, the latter being free to slide vertically in the abutment reaction-detail. This arrangement is shown in Figs. 19, Plate 3.

Carriageway.

The carriageway-width is 40 feet between curbs. In the new structure, the curbs run in straight lines throughout the bridge, but, as the towers must of necessity pass on both sides of the stiffening girders, there is a dead space of approximately 2 feet between the outer edge of the curb and the stiffening girder (Figs. 20, Plate 3). This space is covered by a $\frac{5}{16}$-inch sloping plate, and advantage is taken to provide triangular brackets between the upper edge of the cross girders and the side of the stiffening girder to increase the stiffness of the structure against unequal deflexion of the two stiffening girders. The space under the sloping plate also serves to provide easy access to the carriageway-face of the stiffening girder for the purpose of painting.

The buckle-plates are covered with $\frac{3}{4}$ inch of asphalt laid in two $\frac{3}{8}$-inch coats with stepped joints, the surface of the buckle-plates

being first cleaned and coated with a bituminous paint. Above the asphalt there is a layer of plain concrete 5 inches deep laid in sections with longitudinal and transverse construction-joints. The curb backing concrete is monolithic with the carriageway concrete. The road-surfacing consists of 4½-inch open-jointed wood blocks ("Firmosec" system) laid on the concrete. The curb is laid with 12-inch by 8-inch granite blocks backed up by concrete, and the surface of this concrete between the back of the granite and the lower edge of the sloping plate is finished off to a smooth curve with mastic asphalt.

When the deck was erected, the stiffening girders were not under thrust, and provision was made in the carriageway floor-system as in the lateral system to allow for the stiffening-girder contraction under load. At approximately 94-foot centres, the carriageway steel was broken through to provide a slip joint, and the concrete jointed above this break with a ½-inch bitumen-filled joint running between vertical curb-plates. The wood-block surface, however, was not jointed.

Footways.

The footways are cantilevered outside the stiffening girders (Figs. 21, Plate 3). The handrail runs straight throughout the bridge, and the normal width of the footway is 14 feet. It narrows to 13 feet where the stiffening girders widen out to receive the cables, and to 12 feet at the towers.

The outside face of the stiffening girder is screened from the footway by ¼-inch plates bolted to the girder along their top edges, clamped by 7-inch by ⅜-inch plates along the vertical edges to brackets attached to the stiffening girder and flanged along the bottom edges to a ¼-inch clearance above the footway-edge fillet. This arrangement was adopted to facilitate removal for painting the stiffening girder. The clamping plates are vertical to correspond with the hanger-rods and the balustrade, but the screen-plates are cut square and are standard throughout, except at expansion-details.

To allow for the stiffening-girder contraction under load mentioned in connexion with the carriageways, the footway is broken on a double cantilever at the towers, and the inner line of stringers is not bolted down to the cantilevers but is free to slide over them in clips holding down the toes of the joists. With this in view, also, the footway-stringers at the middle of the centre span were not connected until after the deck load had been transferred to the cables.

The footway-paving consists of mastic asphalt 1½ inch deep laid on a bed of 1-to-6 concrete, which fills the steel troughing and which is finished off at a height of 1 inch above the top of the troughing.

Drainage.

Surface-water from the carriageway is collected by sixteen gullies, 1 foot 6 inches by 1 foot 3 inches by 2 feet 4 inches deep, of galvanized welded steel, placed at the main-span quarter-points, at the towers, at the middle of the side spans, and at the abutments. These gullies are connected by 4-inch outlets to 6-inch diameter cast-iron flanged pipes which run through reinforced holes in the cross girders, and ultimately discharge into rainwater-heads within the abutment-chambers.

To provide for drainage the footway has an outward fall of $2\frac{1}{4}$ inches in its width, and the upper edge of each cantilever is sloped to this fall in order to maintain the minimum depth of footway concrete so as to keep the weight down. The water is collected by a surface channel, 4 inches by $1\frac{1}{2}$ inch, formed in the asphalt paving and underlying concrete, and running along close to the balustrade. This channel is drained by small gullies which are connected to a 4-inch diameter cast-iron pipe, supported on the footway cantilever-brackets, and also discharging into rainwater-heads at the abutments.

Flexible joints are provided in the drainage-pipes where articulations occur in the floor-system, and also at each gully branch.

Demolition of the Old Bridge.

The contract provided that a temporary footbridge for public use should be erected before the traffic on the old bridge was interfered with. This was the first operation to be put in hand. The footbridge was completed in about 4 months, the old bridge closed, and demolition commenced. The same temporary bridge had already been used during the reconstruction of Lambeth bridge, and is again serving a similar purpose at Wandsworth.

The first stage in demolishing the old bridge was the removal of the suspended carriageway and footways. A temporary cable walkway was then suspended under each chain cable and the top cable wedged up off the centre chain cable, where it was dismantled. The second chain cable was then wedged up and dismantled on the lower (or third) chain cable. Finally, for the demolition of the third chain cable its weight was taken on the deck of the temporary walkway, this being effected by tightening up the walkway suspension-cables.

The towers and anchorages, so far as necessary, were removed and the two piers demolished within steel sheet-piled cofferdams.

Generally, the cast iron, masonry and suspension-chains were found to be in good condition, but the wrought-iron carriageway and similar parts had had heavy renewals.

The old bridge[1] had fixed towers, with the chains attached to saddles on roller bearings; a similar arrangement was provided where the chains deflected towards the deep anchorages. None of these bearings showed signs of having functioned for years.

The construction of the piers on timber-pile foundations is of interest. They were entirely encased in cast iron, the lower portion by means of hollow piles, 8 feet apart, driven into the river-bed and interconnected by driven plates 1 inch thick, the joints being formed by interlocking grooves. The upper portion of the casing was in column-and-plate construction of similar section, and finished with a moulded cast-iron coping. The area within the casing had been excavated to hard ground and filled in with concrete to the top of the timber bearing-piles. On this concrete was laid a flooring of stone landings which carried the bases of the towers. The upper portion of the piers was of cellular construction, with a brick lining to the iron casing, and brick diaphragm-walls.

Construction of Piers of New Bridge.

The sheet-piling used for the new cofferdams was Messrs. Dorman, Long's Krupp K III. section, the piles being welded together in pairs and driven from temporary timber stages. The piles were 69 feet long, each pair weighing 2·65 tons. McKiernan-Terry automatic hammers, of type No. 9B2, were used for the first 15 feet or so of driving, and 3-ton drop hammers for the remainder. *Fig. 22* (p. 408) gives a typical record of the driving-resistance met with in the north pier cofferdam.

The Contractors adopted removable steel strutting for the top three frames, and reinforced concrete for the bottom two, the latter being built into the permanent foundation. This method proved satisfactory, and, it is believed, proved an economy to the Contractors.

In spite of the constantly-changing tidal river-levels, leakage was not excessive and could be substantially reduced by dropping ashes from the staging on the river side of the piles. Two sumps were sunk in each cofferdam, and the water-level was kept down by moderate pumping.

When bottom was reached, a drain of small agricultural tiles was laid around the inside of the piling below formation-level, and connected to the sumps. This was concreted over and afterwards grouted-up under a gravity head, through vertical pipes carried up for the purpose above foundation-level.

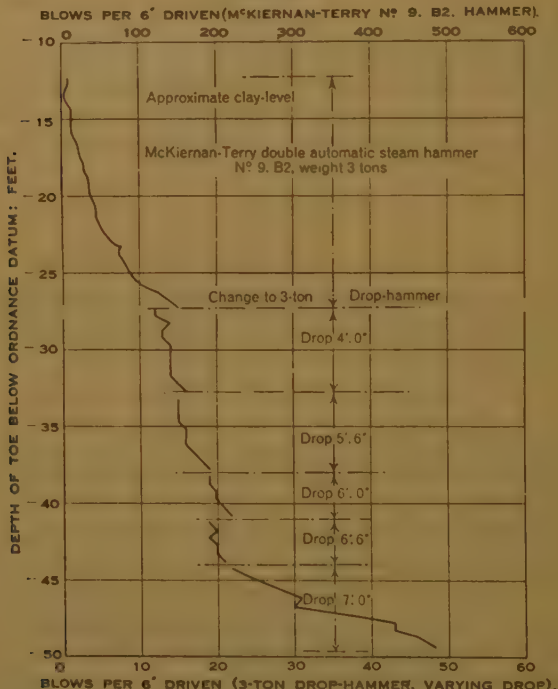
[1] Particulars of the old bridge are given by G. G. Page, "On the Construction of Chelsea Bridge." Transactions of the Society of Engineers, 1863, p. 77.

When the last spit of clay was being removed in certain sections and the first concrete placed, heavy rain occurred, but an examination of the set concrete showed the founding to be perfectly satisfactory, and this was later proved by the behaviour of the loaded pier.

Observations on Rise of Clay in Cofferdams.

During the excavation for the foundation of the north pier, some experiments were made by the Resident Engineer in order to discover

Fig. 22.



Resistance to Drawing of Steel Sheet-Piling for North Pier Cofferdam.

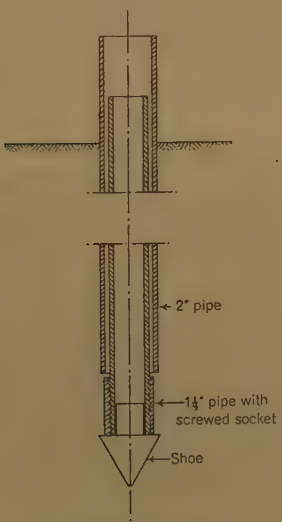
whether there was any tendency for the clay to rise inside the cofferdam as the excavation proceeded.

When a depth of several feet of clay had still to be excavated, two test-pipes were driven to below the bottom level of the foundation, one at each end of the cofferdam. These pipes (*Fig. 23*) were constructed as follows. A conical driving shoe, about $2\frac{1}{2}$ inches in diameter at its base, was fixed to the end of a length of $1\frac{1}{4}$-inch steel pipe. A 2-inch pipe, slightly longer than the $1\frac{1}{4}$-inch pipe, was

slipped over it, like a sleeve, the lower end of the 2-inch pipe resting on the shoe. The twin pipes were driven into the clay to near the desired level. The $1\frac{1}{4}$-inch pipe was then driven about 3 inches further so as to free it from contact with the 2-inch pipe. The smaller pipe would thus be free to record any upward movement of the clay at its foot, without being affected by the clay above that level. Foundation-level was at — 40·0 O.D. The level of the shoe of the pipe at the west end was — 40·2 O.D., and that at the east end — 40·8 O.D.

The depth of clay still to be excavated when the pipes were driven was 8 feet at the west end, and $5\frac{1}{2}$ feet at the east. Observations were taken over a period of 24 days, while the excavation was being

Fig. 23.



(One-eighth full size.)

Device for Measuring Rise of Clay.

completed and the bottom frame of the cofferdam constructed, but practically no movement of the pipes was noted till the excavation was within 1 foot of its final depth.

At the west end a rise of $\frac{1}{8}$ inch was noted at 8 a.m. during the excavation of the last foot; on completion 4 hours later, the pipe had risen $\frac{1}{4}$ inch, and it was then removed. The weather was fair, but rain had fallen the previous day.

At the east end a similar rise of $\frac{1}{8}$ inch was recorded at 8 a.m. before excavation of the last foot was commenced. The excavation was removed between 10 a.m. and 2.30 p.m. At noon the rise was $\frac{1}{4}$ inch,

and this had increased to $\frac{9}{16}$ inch by 5 p.m., when the pipe was removed. Rain had set in during the afternoon.

Further experiments were made by driving 10-inch spikes into the clay close to a step in the last foot to be excavated. Upward movements of from $\frac{1}{16}$ inch to $\frac{3}{16}$ inch were recorded in a few hours. The time available for making observations after the clay was fully excavated was very limited, as, immediately a section of foundation was bottomed, a layer of concrete was put in. The steel piling of the cofferdam extended to about 10 feet below the limit of excavation.

From the fact that the only upward movements recorded occurred after the removal of all but the last foot of clay, these movements would appear to be due to expansion of the surface-layer (probably owing to absorption of moisture) and not to any plastic flow in the mass of clay. Any movement due to increased deflexion of the sheet-piles during the excavation of the last foot would be negligible, and probably non-existent at the axis of the foundation, about 13 feet from the nearest piles, where the pipes were situated.

Deflexion of Steel Sheet-Piling.

Over a period of 5 days, during the time that preparation for the concreting of the fourth frame was in progress, records were taken of variations in the deflexion of the steel sheet-piling at a point 8 feet below the centre of the third frame.

The third frame was at − 17·0 O.D., the clay was excavated to − 28·5 O.D., and the point of observation was at − 25·0 O.D. The measurements were taken across the cofferdam between the faces of the piling, so that the differences recorded were the combined movements of the two sides. The tidal differences varied from $\frac{1}{16}$ inch to $\frac{1}{4}$ inch, and a progressive inward creep was recorded, due to yield in the clay, amounting, at the end of the 5-day period, to $\frac{1}{2}$ inch total or $\frac{1}{4}$ inch on each side.

Settlement of Piers.

Observations were made on the settlement of the piers both during construction and after the bridge had been opened for traffic. For this purpose, vertical pipes $1\frac{1}{2}$ inch in diameter were carried up from the bottom of the foundations, one at each end of each pier. The pipes had 6-inch disks screwed to their lower ends to provide a base. They were placed in position when the first layer of concrete, 1 foot thick, had been laid in the bottom of the foundation, and their upper ends, on which levels were taken from time to time, were just below the top of the cofferdam. The pipes were built in as the work progressed, and when the piers were nearing completion the points

of observation were transferred to the tower anchor-bolts, and also later to the granite bedstones when these had been fine-dressed.

During the construction of the piers the north pier settled about $\frac{1}{8}$ inch and the south $\frac{3}{8}$ inch.

An examination of recorded settlements of other London bridges indicated a probable final settlement of about 1 inch per ton of net pressure per square foot on the clay foundation, and this figure was taken as a guide when deciding what allowance should be made for further settlement. The tower-bedstones were therefore set 1 inch high, the dead-load increment still to be added at this stage being slightly less than 1 ton per square foot. When the bridge was complete and ready for traffic the bedstones were practically at design level, but during the following month a further movement of about $\frac{1}{4}$ inch took place. This increase was probably due more to delayed effect of the dead load than to live-load increment, which was small. The total settlement was thus $1\frac{3}{8}$ inch for the north pier and $1\frac{5}{8}$ inch for the south, and no further movement has been observed. The records of settlement of the north pier during construction are given in *Fig. 24* (p. 412).

In settling, both piers developed a slight tilt upstream. The figures given above are for the west tower-base in each case, the total settlement of the east tower being $\frac{1}{4}$ inch less for the south pier and $\frac{1}{16}$ inch less for the north.

Temperatures of Concrete during Setting and Hardening.

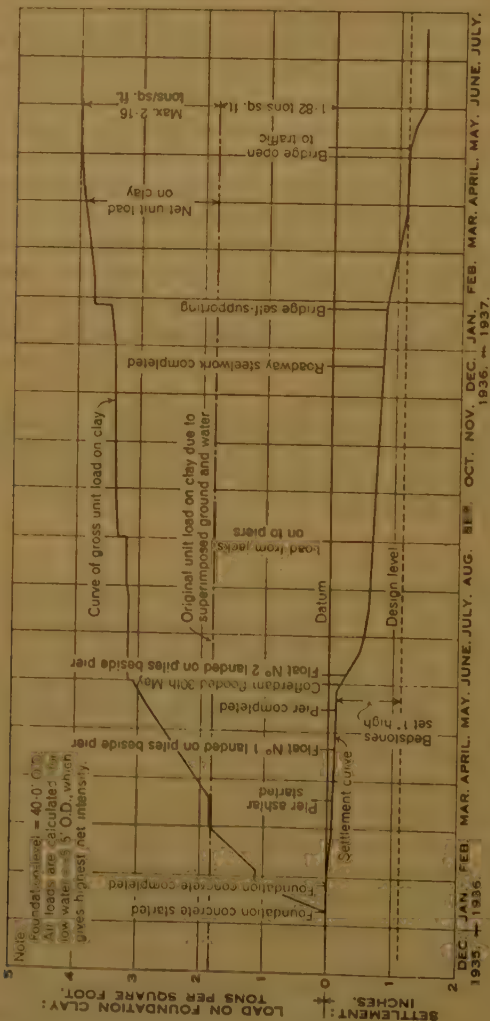
Records of the internal temperatures of the concrete during the period of setting and hardening were taken at certain points in the piers. For this purpose the $1\frac{1}{4}$-inch diameter pipes which had been built in for observing the settlement of the piers were found to be very convenient. Each pipe was filled with grout to the level at which temperature-readings were to be taken; when the grout had set, water was poured in for a depth of about 2 feet and the top of the pipe closed with a cap. Records were taken from time to time by lowering into the water a thermometer carried in an open-topped thin metal tube. This was withdrawn with the thermometer immersed, so that temperature-drop during the few seconds required for reading was reduced to a minimum.

The temperatures taken for the north pier are graphically shown on *Figs. 25* (p. 413), and include one series taken about 6 inches from the surface of the pier. The daily range of atmospheric temperature is also shown, but the temperature inside the cofferdam was maintained by artificial means at a minimum of 35° F. during the construction of the pier. Ordinary Portland cement was used.

Construction of Abutments.

The construction of the abutment river-walls was carried out within steel sheet-piled dams, the front row being driven just in

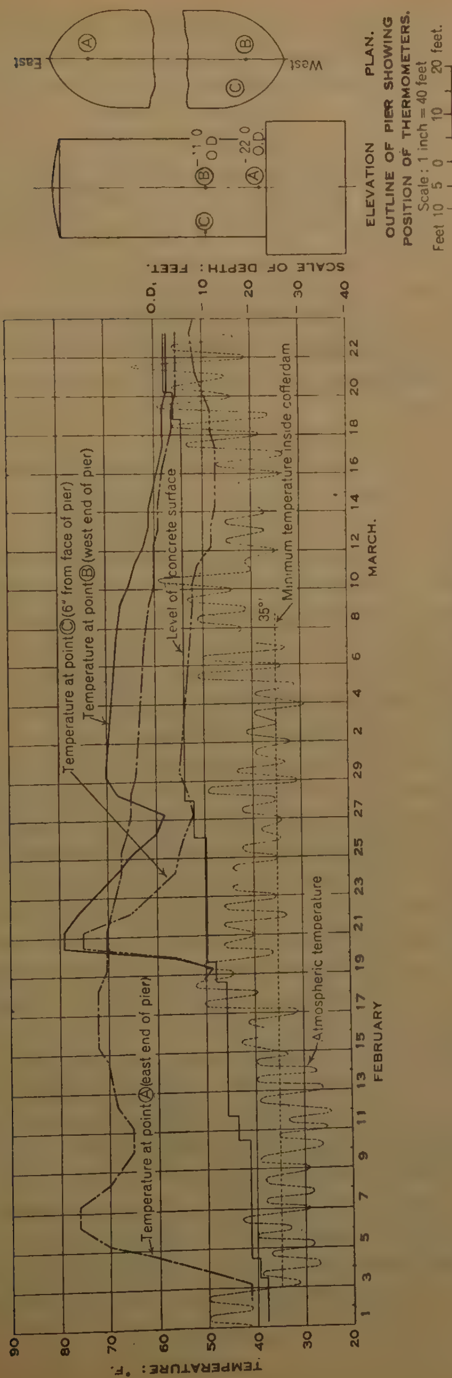
Fig. 24.



Settlement of North Pier under Progressive Application of Dead Load.

front of the old river-walls, and cut-offs being formed by shaped timber piles and clay filling to enclose the area while the river-walls were being cut through and the side and back rows completed. Old

Fig. 25.



INTERNAL TEMPERATURES OF CONCRETE IN NORTH PIER.

masonry was in part demolished by hydraulic cartridges, but this method was found to give blocks too large for convenient handling, and most of the demolition was done with the aid of pneumatic picks.

Pre-mixed concrete was successfully used in the Chelsea abutment as well as in other parts of the work, the concrete materials being transported dry in portable mixers from the suppliers' yard, and mixing commenced on, or just before, arrival at the site.

Fabrication of Steelwork.

Simultaneously with the demolition of the old bridge and construction of the new piers and abutments, fabrication of the steelwork was proceeding.

One of the main considerations affecting the method of fabrication of the stiffening girders was the requirement that the joints should be good butts. Owing to the depth and width of the joints, the weight of the completed sections of girder, and the type of staggered joint adopted, it was not possible to drill all the holes or to machine the joints to gauge from the holes. Further, the length of continuously-butting girder was such that small errors in jig-lengths would tend to accumulate and cause significant errors in overall lengths. A method was developed by the makers in which the various component sections, each section machined to a gauge, were fitted in continuous lengths and then solid-drilled.

After final assembly and riveting, the various component sections did not necessarily lie in one plane at the joint, but each component was a good butt with the corresponding component of the next section. The discrepancy from the flush butt, however, was minimized to such an extent by jig-drilling the pilot-holes for solid drilling and the tack-holes for assembly that riveting could be carried out satisfactorily without disturbing the butts.

In the case of the tower-bases, vertical plates and diaphragms were assembled in skeleton form in the workshop; from this assembly, butt-joints were proved and holes prepared for pilot-drilling. It was then dismantled, taken outside, and completely re-erected in full sections, and drilling and riveting were done while in this position in order to obviate any winding or twisting of the towers.

For the upper sections, the plates for one side were completely laid out to make the butts and prove the full height of the tower, the plates having first been planed from a master plate of each section. Using this assembly as a basis for the fabrication of the towers, the main angles were then attached to the plates with tack-bolts, and, working from one end, sufficient allowance in length was

made to permit of dressing the other ends of the angles to the plates. Divisional units of the base sections were then built up.

The assembly now being a complete section and proved for squareness, the remaining sections followed the same procedure. The plate covers were then attached and drilled in position, and the necessary holes were drilled and plugged to ensure that no movement would take place in subsequent handling. The sections were dismantled and drilled for shop rivets. All members were then re-assembled in complete sections, and diaphragms and angle joint-covers were fixed and drilled in position after all joints were proved to be tight. While in the completely-assembled position, all riveting was done, and the towers were checked both before and after riveting to prove that no winding or twisting had taken place.

Erection of Steelwork.

General Scheme.

In a suspension-bridge of normal type the erection of the superstructure begins with the towers and the cables, as the horizontal tension in the cables is taken by the anchorages. In a suspension-bridge of the self-anchored type, however, it is necessary to complete the stiffening girder before cable-erection is commenced in order to provide the necessary horizontal-reaction points for the cable-tension, and sufficient temporary supports are therefore required to make the stiffening girders self-supporting until such time as their weight can be transferred to the cables. In the case of Chelsea bridge, the side spans, carried on the abutment anchorage-links and on the stiffening-girder bearings at the towers, were of sufficient strength to carry their own weight and that of the carriageway and footway floor-systems without any other temporary supports. The centre span, on the other hand, had to be supported in order to carry the dead load during erection, and, taking into account the navigation-requirements, the simplest and best temporary support was at mid-span.

Any normal method of erection (staging, cantilevering, or flotation) can be adopted for the stiffening girders of a structure of this type, but in the present instance the controlling factor was the necessity for interfering as little as possible with navigation. For this reason, staging would have been unsatisfactory, particularly for the centre span, and as cantilevering would have involved the provision of a large amount of erection-plant, a method of flotation was developed to meet the particular requirements.

A further feature affecting the method of erection was that the stiffening girders in the three spans were, in the final condition, con-

nected together by steel hinges. The various sections of the stiffening girders had therefore to be carefully adjusted for line and level after being placed in position. This requirement necessitated the provision of temporary supports, carrying universal bearings on which the floats could be landed, and the provision of means of adjusting the level of the floats on the bearings by jacks. Also, in order to give sufficient clearance between the section already in position and the next section as it was floated in, making-up pieces were used to close the gap after the floats were placed in position.

Erection-Bay.

The erection-bay was situated between the Battersea pier and the Battersea abutment. The flotation-plant consisted of four barges in two groups of two, connected in pairs side by side. The barges in the flotation condition supported a framework consisting of two main trusses 120 feet long overall and at 50-foot-6-inch centres, arranged to support the stiffening girders directly. Under assembling and riveting conditions the framework rested on six bearings, two at each end and two in the middle, the trusses thus being continuous over three supports. The two main trusses were connected together by eight cross frames. The framework was arranged in such a way as to allow the cross girders to be supported separately from the stiffening girders in the early stages of assembly. The lateral bracing between the trusses was arranged in portal fashion, with two clear spaces over the barge-openings to allow for material being unloaded from barges directly under the flotation-trusses. Cantilever projections of the cross frames on both sides of the main trusses supported the outside stringers of the working platform, 18 feet wide, which extended beneath each stiffening girder at a convenient level for assembling and riveting the deck-structure. Each section of deck as assembled for flotation was 164 feet long overall, and thus projected 22 feet beyond each end of the flotation-framework, and staging supported on timber piling was arranged to carry these projecting portions until the whole deck-section was riveted-up.

The end stagings carried working platforms similar to those on the flotation-framework.

Each of the flotation-truss bearings consisted of a machined spigot bearing resting in a cup-shaped casting with a webbed rectangular base. This type of bearing was adopted to facilitate landing the framework back on its supports after the completion of a flotation.

Packings of 12-inch by 12-inch timber bolted to the top flange of the erection-trusses and to the two end stagings were set to a height corresponding in each instance approximately to the level in the

completed structure of the section of deck under assembly. The precise levels of the camber-packings were made up with steel plates, and were checked frequently during assembly to ensure that neither deflexion of the flotation-framework nor sinkage of the pile supports affected the built camber of the stiffening girders.

Flotation of Sections.

To reduce as far as possible the size of the flotation-plant, the least amount of steel to give the necessary rigidity was floated out into position. The deck structure, as floated out, comprised therefore the stiffening girders (with both components of each girder in position and riveted), and the carriageway cross girders and bottom laterals, together with the carriageway stringers, which although not necessary for rigidity were erected at this stage for progress reasons. The average weight of each float, exclusive of the temporary staging, was about 420 tons.

The deck structure was conveniently divided for flotation-purposes into four sections of approximately equal length, namely, the two side spans and the two halves of the centre span. To meet requirements of navigation, the erection-berth was placed as stated above in the space between the Battersea pier and the Battersea abutment. The Chelsea side span, the north half of the centre span and the south half of the centre span were assembled and floated into position in that order, and the Battersea side span was erected in position on the same flotation-staging.

The only unusual feature of the flotation was that the float was warped out of the erection-bay on the rising tide in the direction of the current, as the working space on the downstream side of the bridge was limited by the proximity of the Victoria railway-bridge. This procedure required extra care in the handling of the float, but the operations were carried out without any untoward incident.

On each occasion a trial flotation was carried out to the extent of lifting the steelwork off its bearings, and the float was trimmed with kentledge (using permanent steelwork not yet erected), the steelwork then being lowered back on to the erection-supports. Previous to this, the shore end of the erection-bay had been dredged to give the required depth of channel. On the day of the flotation (chosen during the spring tides, to give the required rise) the barges were warped in on the lowest practicable state of the tide and the steelwork allowed to lift off its supports on the rising tide. When the float was clear of its bearings, two tugs, coming into position through the trestles of the temporary bridge, took hold at the downstream end, and two more at the upstream end. The float was warped out

parallel with the shore, lateral control being given by the bumping-fenders and control against yawing by the tugs. The average speed of this operation was 8 to 10 feet per minute. When the float was clear of the erection-bay, the warping lines, which had been kept just taut up to this stage to assist in control, were cast off, and the tugs took full control. The float was then hauled across the river into a position opposite the span for which it was intended. Warping lines were again attached and the float pulled as nearly as possible to its correct position. The float was then held by its warping lines until it was possible to bring the hydraulic jacks to bear on the temporary universal bearings. In spite of the motion of the float, no difficulty was experienced in bringing the float to bear without hammering on the bearings. The full dead load of the float was then gradually transferred to the bearings by the falling of the tide. The erection-packings, 1 to 3 feet in height, were then quickly removed and the barges drawn out by the tugs. The barges were anchored in the stream until the next tide, when they replaced the flotation-staging in the erection-bay and the assembly of the next float was proceeded with. The float just placed in position was adjusted to line and level, and the subsequent stages of assembly of buckle-plates, screen-plates, portions of stiffening girder connecting to adjacent sections, footways, etc., were carried out.

Closing Sections.

After the landing of two adjacent floats and adjustment for line and level, the connecting pieces between the floats were assembled in position. Between the side spans and the adjacent halves of the centre span, this making-up piece consisted of a section of stiffening girder 24 feet long and included the tower-hinge pins and the projections connecting to the stiffening-girder bearings at the towers, the section being made temporarily rigid by packings between the upper and lower flanges in the hinge-clearances. At the mid-river support the connecting pieces consisted merely of the centre-hinge pins, which were drawn into position by means of the elongated bolts previously referred to. To complete the main deck-structure, the connecting sections between the ends of the side-span floats and the anchorage-ends were assembled and riveted and the anchorage-ends themselves sufficiently completed to be self-supporting. The weight of the structure was then transferred to the permanent bearings at the abutments and at the towers, the mid-river support in the centre span, however, remaining in operation.

Erection of Towers.

The main tower-bearings were erected in advance of the completion of the adjacent sections of the deck-structure, in order that the stiffening-girder supports at the towers, which rest on the main bearings, might be ready as required when the weight of the deck-structure was to be transferred to the permanent bearings ; the tower side bearings were erected at the same time as the main bearings to facilitate setting all the bearings for one tower as one unit. Following the transference of the weight of the deck-structure to the permanent bearings, the erection of the towers proceeded as was convenient.

Placing of Suspension-Cables.

After the completion of the stiffening girders and the towers, the cables, for which horizontal reaction-points were now provided, were erected, delivery of the ropes from the manufacturers' works being made by road as required. For unwinding each rope, the reel containing it was set up on supports on the south abutment, and one socket secured to the end of a pilot-cable connected to a hand-winch at the other end of the bridge, which pulled the rope across. A 9-inch plank from end to end of the deck provided the necessary runway. The rope-socket was supported by a small wheeled trolley, and wooden bobbins, inserted between the rope and the runway, prevented abrasion. This arrangement was duplicated, so that work on both cables could be carried out simultaneously.

When the rope was unwound, the ends were picked up by two erection-cranes, threaded through the ends of the stiffening girders, and their bearing collars fixed in position. The rope was then lowered to the deck, and the cranes took a fresh hold of it at points opposite the towers and lifted it into its final position, where it was adjusted to its correct marks at the tops of the towers. The lifting device adopted ensured that the rope should not be bent to too small a radius. When all the ropes were in position, the hanger-clips were erected and the hanger-rods attached. The cables at this stage were practically unstressed and were about 15 inches shorter in overall length than in the theoretical geometrical outline taken up under full dead load. For this reason, and because of the deflexion of the various stiffening-girder sections under the erection dead load and the thrust camber, together with the vertical position of the towers adopted for convenience in erection, the holes in the bottom ends of the hanger-rods lay above the corresponding holes in the stiffening-girder hanger-gussets. Various methods might have been adopted to bring these holes into register ; in this case it was found to be

convenient for the towers to be inclined towards the abutments, lowering the cables in the side span and thus allowing the connexion of the side-span hangers to be completed. This movement of the towers, however, raised the centre-span cable and caused an even greater discrepancy between the holes in the hangers and the holes in the stiffening-girder hanger-gussets. The deck-structure had therefore to be lifted in the centre span, which was possible on account of the hinges near the towers and at mid-span. The amount of discrepancy between the positions of the holes was greatest at mid-span, falling to zero at the towers in an approximately parabolic curve. The amount of lift given to each stiffening-girder hanger-gusset hole was, however, approximately proportional to its distance from the tower. It was therefore necessary to raise the centre hinge by about twice the amount indicated by the discrepancy at the centre hanger, in order to allow connexion of the hangers near the towers. A certain amount of downward movement could be obtained in the hangers near the towers by pulling them down by block and tackle, the cable taking a somewhat distorted outline under these conditions.

At this stage the holes in the hanger-rods towards the middle of the centre span lay below the corresponding holes in the stiffening-girder gussets. To complete the connexion of the hangers to the stiffening girder in the centre span, the centre span was now lowered on the jacks at mid-span and the pins inserted as the holes came into register. To facilitate this latter process, the holes in the stiffening-girder gussets had been slotted slightly. The main essentials of the superstructure were now complete, and the further lowering of the middle of the centre span transferred the weight of the deck-system to the cables, and the structure thus became entirely self-supporting.

Erection of Other Steelwork.

The erection of the carriageway buckle-plates, curb-plates and screen-plates, the footway cantilevers, stringers and trough plates and the balustrades was proceeded with in any convenient order.

Costs.

The accepted tender for the work amounted to £310,736. The Contract included the construction and removal of the temporary footbridge and the demolition of the old bridge, as well as the construction of the new bridge and its approaches. The piers cost approximately £38,000, the abutments £37,000 and the steelwork £149,000. Taking the net effective paved area of carriageway and

footways, measured between the faces of the abutments, the unit cost works out at approximately £6·5 per square foot.

Æsthetics of the Old and New Bridges.

The old bridge was interesting in that the functioning parts were masked as much as possible, being considered as something necessarily unsightly that ought to be hidden. The towers were of framed cast iron, not unpleasing in themselves, but they were concealed by cast-iron casing, with glazed lattice panelling. The turning-point supports at the abutments and the chains where they entered the anchorages were also closed in. An ornamental toll-house was constructed at each of the four corners of the bridge, and the whole made as decorative as possible. Before the pinnacles were removed in 1921–22, the structure was a good and effective example of such treatment.

The new structure in comparison is very bare, and depends for any appeal it has upon its general lines, its massing and its simplicity.

Steel in the past has been a difficult material to use æsthetically, although many years ago Gustave Eiffel showed its possibilities when he built the Eiffel tower without disfiguring a beautiful city. The steel pylons carrying the electric transmission-lines of the British grid system are further examples of steel at its best, whereas Charing Cross bridge is an eyesore.

The recent trend in taste towards cutting out the purely ornamental and depending more for a pleasing effect upon the proportioning and design of functional members, together with the wider use of welding and the improvement in paints and metallic sprays, makes functional steel structures more acceptable now than in the past.

Although the new bridge is unadorned, more time was probably spent on its æsthetics than on those of the old one. In this respect due regard was paid throughout to the views of the Royal Fine Arts Commission.

So far the new bridge has merely been accepted, and it remains to be seen if its simplicity has any real appeal.

Conclusion.

The old bridge was demolished and the new bridge was completed within the estimated cost, and was ready for opening 5 months before the date for completion, just in time to take the heavy traffic for the Coronation. It was opened on the 6th May, 1937, by The Right Hon. W. L. Mackenzie King, C.M.G., Prime Minister of the Dominion of Canada.

Everything went smoothly from start to finish, owing to the close and friendly collaboration that existed throughout among those concerned with the work.

A list of some of the many who were concerned in the construction of the bridge is given in Appendix I.

The Paper is accompanied by twenty-seven sheets of drawings and two photographs, from some of which Plates 1, 2, and 3, the Figures in the text, and the half-tone page plate have been prepared, and by the following five Appendixes.

APPENDIX I.

PERSONNEL.

London County Council :—

The Right Hon. Lord Snell, C.B.E.	Chairman.
The Right Hon. Herbert Morrison, J.P., M.P.	Leader of Council.
Mr. G. R. Strauss, M.P.	Chairmen of Highways Committee.
Mr. R. Coppock	Chairmen of Highways Committee.
Mr. T. Peirson Frank, M. Inst. C.E.	Chief Engineer.
Mr. K. T. Lomas, M. Inst. C.E.	Assistant Chief Engineer.
Mr. H. Firth, Assoc. M. Inst. C.E.	Divisional Engineer.
Mr. H. G. Lloyd, M. Inst. C.E.	Assistant Engineer.
Mr. G. Topham Forrest, F.R.I.B.A.	Late Architect.
Mr. E. P. Wheeler, F.R.I.B.A.	Architect.
Mr. J. Maxwell Scott, A.R.I.B.A.	Assistant Architect.

Royal Fine Arts Commission.

Contractors :—

Messrs. Holloway Bros. (London), Ltd.

Mr. Henry Holloway	Chairman.
Mr. Roland Holloway	Director.
Mr. W. Storey Wilson, B.Sc., M. Inst. C.E.	,,
Mr. A. E. Reid, Assoc. M. Inst. C.E.	Agent.
Mr. P. W. E. Holloway, Assoc. M. Inst. C.E.	,,
Mr. H. Wood	General Foreman.

Sub-Contractors :

The Furness Shipbuilding Co., Ltd.	(For all steelwork, fabricated.)
Messrs. Wright's Ropes, Ltd.	(For suspension-cables.)

Resident Engineer's Staff :—

Mr. Duncan Kennedy, M. Inst. C.E.	Resident Engineer.
Mr. L. A. Travers, B.Sc., M. Inst. C.E.	Assistant Resident Engineer.
Mr. N. G. Sisson	Assistant.
Mr. J. Angus	Clerk of Works (Granite).
Mr. W. Alvey	Clerk of Works (Steelwork).
Mr. W. H. Holliwell	Inspector.
Mr. A. W. Cuthill	,,
Mr. P. M. Pascoe	,,

Engineers—Messrs. Rendel, Palmer and Tritton :—
Mr. A. T. Best, M. Inst. C.E.
Mr. A. J. Clark, B.Sc., Assoc. M. Inst. C.E.
Mr. J. R. H. Otter, B.Sc., Assoc. M. Inst. C.E.
Mr. E. O. Measor, B.Sc., Assoc. M. Inst. C.E.
And the Joint Authors.

APPENDIX II.

Further Particulars of High-Tensile Steel.

A manganese-copper steel was used for the 105½-inch-wide webs, and a manganese-chromium-copper steel for the remainder of the stiffening girders. Both steels were specified to have an ultimate tensile strength of 37 to 43 tons per square inch with a minimum yield-stress of 23 tons per square inch and a minimum elongation of 18 per cent. on British Standard Test Piece " A." For design purposes a working stress of 12·65 tons per square inch was allowed. In view of the comparative newness of the steels adopted, it was specified that approximately 2½ times the number of tests specified for mild steel should be taken, in order to establish the uniformity of the material and to provide data as to the comparative desirability of plain manganese or chromium-manganese steels. Tables II and III give a summary of the results obtained.

Table II.—High-Tensile Steel.
Percentage Chemical Composition.

		C.	Si.	S.	P.	Cu.	Mn.	Cr.
Specified		0·30 (max.)	0·20 (max.)	0·05 (max.)	0·05 (max.)	0·30 to 0·60	—	—
" Ducol " (Colvilles, Ltd.)	Range.	0·24 to 0·26	0·10 to 0·15	0·02 to 0·03	0·03 to 0·04	0·35 to 0·38	1·40 to 1·67	—
	Average of all tests.	0·25	0·13	0·03	0·03	0·36	1·52	—
"Atlantes " (Cargo Fleet Iron Co.)	Range.	0·22 to 0·28	0·05 to 0·17	0·03 to 0·04	0·02 to 0·04	0·40 to 0·53	0·74 to 0·98	0·40 to 0·48
	Average of all tests.	0·25	0·12	0·04	0·03	0·43	0·92	0·44

In order to simplify the jointing of the girders, it was decided that a corresponding high-tensile rivet-steel should, if possible, be obtained. Several of the larger steel-companies had been experimenting on this problem for some years, and had produced steels which would give satisfactory riveting qualities and a driven shear-strength 30 to 50 per cent. greater than normal mild-steel rivets.

TABLE III.—HIGH-TENSILE STEEL.

Physical Properties.

		Yield-stress: tons per square inch.	Ultimate tensile strength: tons per square inch.	Elongation: per cent.	
				Longitudinal.	Crosswise.
Specified		23 (min.)	37 to 43	18 (min.)	18 (min.)
"Ducol" (Colvilles, Ltd.)	Range.	23·0 to 27·9	39·2 to 46·0	18·0 to 26·0	18·0 to 24·0
	Average of all tests.	24·84	41·46	21·5	20·98
"Atlantes" (Cargo Fleet Iron Co.)	Range.	23·3 to 28·0	37·1 to 41·6	18·0 to 25·0	18·0 to 28·0
	Average of all tests.	25·02	39·21	21·07	21·04

Various types have been tried, including plain manganese and manganese-chromium alloy-steels, with differing percentages of the alloying materials. The suppliers of the material carried out tests on the rivet-bars and undriven rivets for tensile strength and elongation, and on the driven rivets for shear, tensile strength and elongation, and supplied chemical analyses and material for check-analyses. Two casts of rivet-material were subjected to tests, and the results are given in Table IV.

TABLE IV.—TENSILE TESTS ON RIVET-BARS.

	Dimensions of test-piece.		Breaking.		Elongation: per cent.	Dimensions after breaking.		Reduction: per cent.
	Diameter: inches.	Area: square inches.	Load: tons.	Stress: tons per square inch.		Diameter: inches.	Area: square inches.	
1st cast material.	0·87	0·594	17·70	29·8	34	0·49	0·189	68·2
2nd cast material.	0·875	0·601	19·30	32·1	31	0·525	0·216	64·0

The results of tests on driven rivets from the same two casts are given in Table V.

The minimum driven shear-strength was specified to be 26 tons per square inch, and as it was not feasible to carry out routine tests on the material during production, owing to the handling of the material by the mills and rivet-manufacturers before such tests could be made, it was considered desirable to

TABLE V.—SHEAR TESTS ON DRIVEN RIVETS.

No. driven together.		Dimensions of test piece.			Breaking.	
		Diameter: inches.	Area: square inches.	Total area: square inches.	Load: tons.	Stress: tons per square inch.
1st cast material	1	0·916	0·659			
		0·916	0·659	1·138	33·6	25·5
	1	0·925	0·672			
		0·918	0·662	1·334	33·5	25·0
	2	0·918	0·662			
		0·920	0·665	1·327	32·5	24·5
	2	0·916	0·659			
		0·921	0·666	1·325	33·2	25·0
	3	0·920	0·665			
		0·920	0·665	1·330	35·5	26·6
	3	0·914	0·656			
		0·914	0·656	1·312	32·1	24·5
2nd cast material	1	0·934	0·685			
		0·931	0·681	1·366	39·4	28·8
	1	0·929	0·678			
		0·931	0·681	1·359	37·6	27·7
	2	0·935	0·687			
		0·937	0·690	1·377	39·5	28·7
	2	0·938	0·691			
		0·937	0·690	1·381	37·8	27·4
	3	0·926	0·673			
		0·927	0·675	1·348	38·0	28·2
	3	0·934	0·685			
		0·927	0·675	1·360	37·6	27·6

specify the tolerance on chemical analysis. The range of alloying ingredients allowed was based on the tests carried out by all the manufacturers, and the upper and lower limits of chemical analysis specified, together with the probable physical properties of the steel produced to these limits, were as follows:—

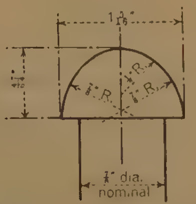
Carbon	0·16 to 0·22 per cent.
Silicon	0·10 to 0·20 ,,
Sulphur	0·05 per cent. max.
Phosphorus	0·05 ,,
Copper	0·29 to 0·39 per cent.
Manganese	0·75 to 0·85 ,,
Chromium	0·29 to 0·39 ,,

Tensile strength of rivet rod . . .	30·9 to 34·5 tons per square inch.
Shear strength of driven rivet . .	26·2 ,, 31·3 ,,
Tensile strength of driven rivet .	37·1 ,, 46·6 ,,

Elongation on 2-inch gauge of driven rivet, 26 to 19 per cent.

The shape of the head of high-tensile steel rivets before driving is shown in *Fig. 26*.

Fig. 26.



This shape was chosen to distinguish between rivets of high-tensile steel and mild steel, and to ensure better filling of the rivet-hole, the finished head being formed with a standard snap. The rivet-heads are similar to those used on the Sydney Harbour bridge.

APPENDIX III.

Further Particulars of Steelwork.

Stiffening Girders. The main diaphragms between the two component girders consist of two main vertical plated diaphragms at cross-girder points, an 11-inch by ⅜-inch plate joining the top flanges together, and a batten-system joining the two bottom flanges. Subsidiary vertical plated diaphragms are arranged between the girders at approximately the third-points between the cross girders, and in addition a horizontal plated diaphragm runs between the vertical diaphragms.

Further stiffening is provided by vertical angles fitted between the top and bottom angles on the outside of each girder, midway between all vertical diaphragms, and by horizontal angles at approximately the quarter-points of the depth of the girder on the inside, running between the vertical diaphragms.

All diaphragms other than the main vertical diaphragms at the cross girders are provided with manholes. In the wide section of the stiffening girder the diaphragm plates are stiffened by angles.

The joints in the girders occur at intervals of approximately 31 feet 3 inches. Both component girders are jointed at the same point, and each joint is symmetrical.

As anticipated, difficulty was experienced in keeping within the usual tolerance in the manufacture of the web-plates 105½ inches wide, and it was found that in order to obviate a large proportion of rejections, a tolerance of 7½ per cent. in the thickness was necessary with a minimum thickness of − 2½ per cent. No difficulty was experienced in making up the girder with these plates.

Hangers. The hanger-clips (Figs. 8, Plate 1) are 2 feet 7½ inches long, and are made in two halves which are connected by a row of bolts above and below the cable. At each hanger-clip, two cast-steel wedge-pieces 1 foot 3 inches long are inserted within the cable as an additional precaution against slipping.

In tests to prove the efficacy of these wedges, carried out with two hanger-clips on a 7-foot length of cable using a 1-foot 11-inch spanner for bolting up,

slip commenced when a sliding force of 52 tons was reached, but 63 tons was held after a total slip of ½ inch had occurred.

Before bolting up the hanger-clips, hanger-gussets 2½ inches thick were inserted, varying in shape with the angle which the cable makes with the vertical hanger-rod. The details of the hanger-rods are shown on Figs. 8, Plate 1. Where the distance between the cable and the stiffening girder is too small to permit hanger-rods with the conventional eyes to be used, and where the cable is below the top of the stiffening girder, two flats are used instead, passing through slots in horizontal diaphragms and connected by pins to diaphragms arranged just above the bottom flange of the girder.

Anchorage-ends. Under the end cross girder a deep trussed girder with its bottom flange connected to the lowest point on the anchorage is arranged to maintain the two stiffening girders against twisting.

The connexion of the rope-sockets to the end of the anchorage is made through split castings (Figs. 9, Plate 1) located by a spigot fitting into a hole in the bearing plate, and bearing on pairs of diaphragms riveted between the side-plates of the anchorage-ends. At the steel-fabricators' suggestion, the 8-inch by 6-inch by ⅝-inch end angles were bent to a true circular curve, and the 1-inch bearing plates were bent to suit and then machined off square on the face bearing on the seating castings which were themselves machined. The side-plates of the anchorage-ends were kept back from the heel of the end angles, and the individual bearing diaphragms machined and finally ground off to bear on the bearing plates.

APPENDIX IV.

Plant.

As far as practicable, all plant was electrically driven, the power being generated on the site.

A central compressor-plant provided air for riveting and other pneumatic tools, and when this had to be closed down for reasons of progress, portable compressors were employed.

The principal cranes used for demolition and construction were seven in number, all of derrick type. Two of 14-ton capacity were erected on each pier staging, upstream and downstream, one 5-ton on the north abutment, one 7-ton on the south abutment, and one 10-ton adjoining the erection-bay in the south side-channel.

Concrete-mixing was chiefly done by floating equipment which also batched the aggregate and cement. This arrangement provided a mobile means of concreting at different parts of the work as required, and also relieved to some extent the land area, necessarily limited, which was available for the Contractor's operations.

APPENDIX V.

Time-Table.

Date for commencement of Contract	15 October, 1934.
Staging commenced	October, 1934.
Temporary footbridge commenced	November, 1934.
,, ,, opened	4 March, 1935.
Old bridge closed to vehicular traffic	28 February, 1935.
,, ,, pedestrian traffic	4 March, 1935.
Demolition of old bridge commenced	1 March, 1935.
,, ,, ,, completed to pier level	31 May, 1935.
Pier cofferdams commenced	June, 1935.
Concreting of south pier foundation commenced	23 November, 1935.
,, north ,, ,, ,,	9 January, 1936.
South pier completed	April, 1936.
North pier completed	June, 1936.
Erection of deck-steelwork commenced in erection-bay	9 March, 1936.
First section of deck-steelwork (north side-span) floated into position	26 April, 1936.
Second float (north half of main span)	7 June, 1936.
Third float (south half of main span)	12 July, 1936.
Fourth section erected (south side-span)	September, 1936.
Towers completed	November, 1936.
Cable erection commenced	19 November, 1936.
,, completed	18 December, 1936.
Fixing of hanger-clips completed	16 January, 1937.
Hanger-connexions completed and bridge made self-supporting	25 January, 1937.
Upstream footway opened to public and temporary footbridge closed	2 February, 1937.
Demolition of temporary footbridge commenced	4 February, 1937.
Paving of bridge and approaches completed	1 May, 1937.
Bridge opened to public	6 May, 1937.
Date for completion of Contract	15 October, 1937.

Discussion.

The President. The PRESIDENT, in moving a vote of thanks to the Authors, s that he had read the Paper with great interest. The simplicity the design was admirable, and ornamentation would not have m the bridge more meritorious than it was. He felt that it could called a masterpiece of simplicity.

Mr. Buckton. Mr. E. J. BUCKTON said that, unfortunately, Mr. H. J. Feree was unwell, and was unable to attend the Meeting. He exhibited number of lantern-slides illustrating the work described in the Pap

Dr. Anderson. Dr. DAVID ANDERSON observed that there had been no previ examples of the self-anchored suspension-bridge in Great Brita he envied the Authors the opportunity that had fallen to them introducing the new type of construction with the numerous det which had had to be considered, and their courage in taking it.

Had the Authors made any further studies in connexion with self-anchored type of suspension-bridge, and, in particular, had th formed any idea of the economic limiting span of such a bridg His firm had made two studies in connexion with that type of str ture. One had been to assist a contractor with the technical side his tender. The contractor had been engaged in an internatio competition, and the span involved was about 800 feet, rather m than double the span at Chelsea. As far as they had been a to make out, the adoption of the self-anchored type would have to an economy lying somewhere between 13 and 25 per cent. T other study which they had undertaken was for a span of 1,500 fe and there undoubtedly the self-anchored type was not economic the large amount of steel which would have had to be put into stiffening girder when it acted as a strut quite outweighed a economy which could have been made in the anchorages. He wou put the limiting span at about 1,000 feet, which was the span wh was adopted for Mülheim bridge over the Rhine. It would be helpf however, if the Authors would say how much was saved in the c of the Chelsea bridge by comparing the cost of the additional steel the stiffening girder acting as a strut with the cost of anchorages the ordinary type. There were naturally occasions where anchorage of the ordinary type was very expensive, as for instan in bad ground, and then the self-anchored type of structure was ve attractive; but in the case of Chelsea bridge the Authors were deal with more normal conditions—ballast overlying London clay—a presumably an anchorage of the ordinary type would not be ve

Dr. Anderson.

xpensive. Another point in connexion with the self-anchored type hich was referred to in the Paper was that in many instances fairly xpensive works would have to be adopted to carry the stiffening rder temporarily before it was supported by the cables.

With regard to the settlement of the piers, the Authors had given very interesting diagram (*Fig. 24*, p. 412) showing that settlement ad hardly begun at all until a very large percentage of the total eight had been put upon the foundations. They stated that, from e average which they had been able to determine from other ondon bridges, they expected the settlement to be of the order of inch per ton per square foot of net additional load on the clay. the early part of the Paper it was stated that for the Chelsea ridge the additional load would be in the neighbourhood of $2\frac{1}{2}$ tons er square foot, so that, on the basis of a settlement of 1 inch per ton er square foot of net additional weight, a settlement of $2\frac{1}{2}$ inches ould have been expected; *Fig. 24*, which apparently gave the otal settlement, showed that it was about $1\frac{1}{4}$ inch. It would, herefore, be of interest to know whether the Authors considered hat the figure of 1 inch per ton per square foot, deduced from apers on other London bridges, was rather high.

Sir Benjamin Baker had been concerned with the question of the ettlement which took place in London clay when Blackfriars bridge as being widened. The piers were being lengthened by building new masonry in course with the old masonry, and, rather than sk any settlement, all the piers had been test-loaded, which had een fairly costly. If it had been possible to predict closely before-and what the settlement would have been, it might have been ossible to save that expenditure. Actually, the net additional load t Blackfriars was about 3 tons per square foot, and the settlement as $1\frac{1}{4}$ inch, compared with the 3 inches given by the average figure entioned in the present Paper.

Dr. Anderson had not had much experience of pre-stressed cables, ut he had been informed by a cable manufacturer that if, after cable had been pre-stressed, it was wound on a drum and then nwound, the advantages gained by the pre-stressing were very argely lost. That was a very serious statement, and he would like o know whether or not in the opinion of the Authors it was true. n the case of the bridge over the Rhine to which he had referred, e believed he was correct in saying that the Germans had evidently ad some such idea in mind, because the pre-stressing had been one alongside the bridge and the cables had been dragged up into osition without being put upon drums.

Mr. H. T. Holloway.

Mr. H. T. Holloway welcomed the opportunity of saying that he contractors were very glad that they had brought the work to a

Mr. H. T. Holloway.

successful conclusion. The President had commented on æsthetic quality of the bridge, and it might interest the engin present to know that several leading architects who were friend Mr. Holloway had without exception expressed their admiration the structure from the æsthetic point of view. The relations of contractors with the engineers had been most happy from the co mencement of the work, and they had always found the Authors their engineers most helpful in checking their calculations.

Mr. Firth.

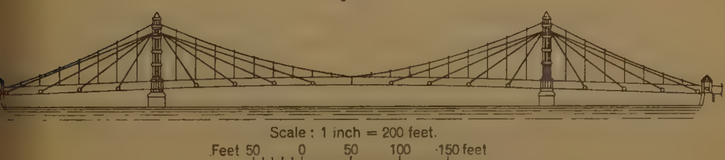
Mr. Harold Firth said that the old Chelsea bridge, with its restriction of 5 tons, had long been regarded as a weak bridge fact, it was doubtful whether it had ever been strong enough fo work. Very soon after it was opened in 1858 questions had ar as to its strength, and the Office of Works, for whom the bridge been constructed by Mr. Page, had asked an independent engin Mr. Edwin Clark, to advise them. He had reported that the bri was not as strong as a Metropolitan bridge should be, and a result of that the Office of Works had consulted the then Presid of The Institution, Sir John Hawkshaw, who had recommended t the bridge should be strengthened at once. Certain works had b carried out in accordance with his recommendations in 1863 1864, at a cost of about £11,000. After the bridge had been p chased by the Metropolitan Board of Works in 1879 and freed fr tolls, Sir Joseph Bazalgette had reported on its strength and recommended further strengthening, which consisted of the additi of a third suspension-chain and alterations of the suspension-ro costing a further £11,000. In 1903 it had been necessary to rep defects in the deck-plates and certain cross-girders, and ag in 1929 and 1930 there had been rather extensive repairs to deck-plates and cross-girders, chiefly near the abutments and pie the spires on top of the towers had to be removed about 19 When, therefore, the chief engineer of the London County Cou had reported that unless the bridge was reconstructed at an ea date more extensive repairs to the decking would be necessary, Council had decided in 1931, on his report, to proceed with reconstruction of the bridge. At that time the Council had thou that it should be a six-line bridge; but, as was stated in the Pap the work had been deferred on account of the financial crisis, and was not until November 1933 that the Council had decided to proce and, after consulting the Ministry of Transport, they had decided make it a four-line bridge.

With regard to the type of bridge adopted, the Authors stated the Paper that "The self-anchored type of suspension-bridge adop has been used in various places on the Continent and in Amer where site-conditions made it undesirable to subject the anchora

horizontal loads." Dr. Anderson also was evidently not aware Mr. Firth.
the fact that there was another bridge of that type in England; amely, the Albert bridge, the next bridge to Chelsea bridge on the ostream side. Its main members consisted (*Fig. 27*) of flat bars diating from the tops of the towers and connected to the parapet-rders, but above the radial bars there was a suspension-chain which so supported the radial bars and the parapet-girders by vertical ars; that suspension-chain was connected to the ends of the parapet-rders, which also acted as the stiffening girders for the suspension-stem, and the chains were anchored to the abutments by vertical ak-bars. In so far, therefore, as it was a suspension-bridge, it was early of the self-anchored type.

He had visited the works on several occasions during the con-ruction of the new bridge, and one thing which had impressed him as the efficiency of the steel sheet-pile cofferdam used for the nstruction of the piers. When he saw the foundations, the bottom

Fig. 27.



as as dry as could be wished. It had been about that time that the ief engineer of the London County Council had been considering hat type of foundation should be adopted for the new Wandsworth idge, and it had been decided to give the tenderers the opportunity tendering either for caisson foundations sunk under compressed r or for open cofferdams. The lowest tender showed a saving about £12,000 in favour of the cofferdam type of foundation, d the chief engineer had recommended the London County uncil to proceed on that basis. The excavation for the north pier undation at Wandsworth bridge had recently been completed side a steel sheet-pile cofferdam, very similar to that used at elsea bridge. It was specified that the lower frame should be nstructed in reinforced concrete, and an almost perfectly dry fferdam was obtained by catching any water leaking through the eet-steel piling at that reinforced-concrete frame.

He was particularly interested in the figures given on p. 410 with gard to the deflexion of the steel sheet-piling. It was stated on 410 that, when it was left unsupported for a depth of 11 feet 6 inches ring a period of 5 days, there was a deflexion of ¼ inch on each side.

Mr. Firth.

There had been considerable argument with the contractors Wandsworth bridge on the subject, and the figures given in Paper seemed to prove clearly that the clay was exerting an ac pressure on the steel sheet-piling. It would be of interest if Authors could furnish any further information on that point.

With regard to the settlement of the foundations, he agreed Dr. Anderson that the figure of 1 inch per ton of pressure per squ foot seemed rather excessive, and hardly agreed with the settlem actually recorded and shown in *Fig. 24*, p. 412, which clearly sho that the settlement of the clay did not take place immediately on application of the load, but that there was a certain time-lag. T was shown by the acceleration in the rate of settlement during J 1936, and in February and March, 1937. It might be of interes refer in that connexion to some loading tests made on foundat for the pier-extensions to carry the widened Putney bridge, a so what similar work to that which Dr. Anderson mentioned at Bl friars bridge. In the case of Putney bridge the settlement und total load of 4 tons per square foot was only about ½ inch, and w the test load was removed, reducing the pressure by about 2·7 per square foot, there was an upward movement and the recovered by about ¼ inch. In that case, however, it was poss to leave the test loads in place for only the comparatively s period of 7 days, during which very slight settlement was obser Had it been possible to leave the test load on longer, it was ceivable that more settlement would have occurred.

Mr. Storey Wilson.

Mr. W. Storey Wilson observed that three main problems faced the contractors at the commencement of the work: the dem tion of the old bridge; the design and construction of the co dams; and the floating-in of the spans. He would like to say a words about each.

The temporary suspension-footbridges for the demolition of old chains had been erected under each set of suspension-ch before the removal of the bridge-deck, and not, as the Authors sta afterwards. It had been quite a simple matter to erect those bri while the deck was still intact. The deck-system of the old br had been removed piece by piece by four hand-cranes, with the of oxy-acetylene burners, commencing at the centre of the br and at both abutments and working towards the towers, in orde preserve the balance. The whole deck had been removed in working days. With the removal of that load, some of the sag been taken out of the chains, and it was interesting to note that two lower chains pushed up the top chain to such an extent th buckled at the pins, thus showing that it had not been carrying same load as the two lower chains. It would appear that when

ird chain was added it had been hung at the required distance Mr. Storey Wilson.
oove the second chain and had been duly connected through links
the hanger-rods. It was probable, therefore, that the third chain
ad at no time carried more than its own dead load, except, perhaps,
ich small load as might have come upon it by the stretch of the
wer chains under live load. That had been confirmed by the fact
at, while the bridge was still under traffic, the period of vibration
 the top chains was much slower than that of the lower chains.
he chains, which weighed approximately 500 tons, had been
moved in a period of 9 working days in the manner described by
ie Authors.

Analyses and tests of the material taken from the chains of the
d bridge had given the following results :—carbon 0·024 per cent.,
licon 0·156 per cent., sulphur 0·015 per cent., phosphorus 0·245 per
nt., manganese 0·057 per cent., chromium 0·037 per cent.;
ltimate tensile strength 22·2 tons per square inch; yield-point
5·1 tons per square inch; and elongation on a 2-inch gauge-length
9·5 per cent.

In discussing the type of pier-foundation adopted, the Authors said
at the choice lay between caissons and open cofferdams; when, how-
ver, it had been decided to build the new piers on the site of the old
iers, there had been really no choice, for he could scarcely imagine
ny engineer contemplating sinking caissons over old piers in prefer-
nce to open cofferdams. The contractors had designed the coffer-
ams, which were 59 feet deep, and had provided for five frames, the
p three being in steel and the bottom two in reinforced concrete.
he small number of frames and the distance between struts, which
ere spaced 10 feet apart, greatly facilitated the demolition of the old
iers and the building of the new piers. The lower concrete frame
as cast up against the sheet-piling and was converted into a
rainage-channel. All the water running down the face of the piling,
nstead of finding its way to the bottom of the cofferdam, as was
sually the case, was caught on the concrete frame and was carried
ff in pipes to the sumps. The result of that arrangement was that
he bottom of the foundation, when prepared to receive the first lift
f concrete, was exceedingly dry; in fact it was so dry that the
uthors had commented on the effect of rain when the last spit of
lay had been removed. In most of the cofferdams with which he
nd others had had to deal, there had always been too much water
resent to give rise to fears about the effect of rain. The Authors
ad expressed the opinion that the design of dam had proved to be
n economy to the contractors, but Mr. Wilson suggested to con-
ractors generally that that opinion should be accepted with reserve.
t might, however, be advantageous in other ways; the work was

Mr. Storey Wilson.

undoubtedly speeded up, and probably earned the contractor a bo for early completion. The cost of the work actually carried inside the cofferdam was probably lower because the obstructi caused by steel and concrete frames were less than would h occurred with timber frames. Work was cleaner and drier, and setting-out and inspection could be carried out in greater comfor

Looking back and considering the opinions expressed on for occasions as to the relative merits of caissons and open cofferd for the construction of the river-piers for Thames bridges, it was claiming too much to say that the results realized at Chelsea wo have an important bearing on the choice to be made for fut Thames bridges. The piers for Wandsworth bridge were at pres being constructed in open cofferdams at considerable saving in c over caisson foundations, as Mr. Firth had remarked, and it probable that the Chelsea cofferdams had had some influence in decision in that case.

The floating-in of the spans was perhaps the most difficult operat in the whole of the bridge-construction. The contractors' design the floating-in staging described in the Paper, and the detail arran ments which they had had to make, required the greatest care view of the short duration of a tide and the narrow limits wit which they had to work. They had been fortunate in obtaining use of four eminently suitable pontoons from the Port of Lon Authority. During the carrying-out of the contract, the Port London Authority had given the contractors every facility. T Battersea arch had been closed to river traffic, and the Chelsea a had been very much restricted. In return, the contractors had u every possible means to prevent any dislocation of river traffic, a he did not think that there had been a single complaint on th account during the whole period of the contract. The contract gave due credit to the Furness Shipbuilding Company for supplyi the steelwork in such good shape and time as to enable the bridge be opened to traffic 23 weeks ahead of contract time.

Mr. Lloyd.

Mr. H. G. Lloyd remarked that he had been engaged in testi materials in connexion with the bridge and in keeping tidal recor With regard to tide-levels, the Newlyn datum was not mentioned the Paper; as that was the accepted datum for record work in t Thames, it might be as well to state that at Chelsea bridge t Newlyn datum was 1·33 foot above the Ordnance datum. Th difference did not apply, however, to other parts of the Tham Tidal records had been kept at Chelsea since 1883. During t construction of the new bridge, the tide-gauge on the southern Surrey pier was transferred firstly to the temporary bridge, gre care being taken to secure accurate records, and finally to the n

uth or Surrey pier. The pier contained a vertical pipe 14 inches Mr. Lloyd. internal diameter, connected to a horizontal pipe placed at - 10 N.D. The tide-gauge was connected to a 12-inch diameter oat, and the record was transferred to a diagram which was on a rger scale than the previous one and corresponded with that used y the Port of London Authority.

He had carried out a number of tests on samples of clay, which ere taken from the pier-foundations as soon as the clay was xposed and were tested by means of a double shearing test. hat test gave the values of K (the initial intensity of resistance shear) and of α (the angle showing the rate at which the crease in resistance to shear with increased normal pressure took lace). Thence, by means of a formula for which he was responsible,[1] was easy to calculate fairly rapidly the load which the clay at any ne level would support. The clay in question had been found to be f a different quality from that found under pier No. 5 of Waterloo ridge, which had failed. The greatest value of α was 21 degrees 8 minutes, and the lowest was 10 degrees 57 minutes, the latter gure corresponding more nearly to that obtained at Waterloo bridge. he maximum figure for K was 1·553 and the minimum 0·59 ton per quare foot. The clay itself varied in density from 124 to 130 lb. er cubic foot, the depths whence the samples had been taken anging from — 17·50 to as low as the actual foundations, — 39·50 .D. From those figures, calculations had been made of the oad which the clay would withstand, by means of the formula which he had referred. The results varied, according to the osition of the clay, from 5·8 to as much as 13·9 tons per square foot.

Various tests had been made on the concrete. In particular, ompression-tests had been carried out with an hydraulic press, sing two gauges on the press to ensure accuracy. Concrete with roportions of 6 to 1 by volume of ballast and Portland cement, atured in air, gave an average crushing strength of 2,631 lb. per quare inch at 7 days, and 3,668 lb. per square inch at 28 days, the ensity being 145·9 lb. per cubic foot at 7 days, and 144·5 lb. per ubic foot at 28 days. A remarkable result was obtained with what ight be called an ideal mix of $1 : 1\frac{1}{3} : 2\frac{2}{3}$, a crushing strength of ,067 lb. per square inch being obtained at 7 days with a density of 49 lb. per cubic foot.

Other materials employed had also been tested ; a test carried out n some of the asphalts was perhaps of special interest. In order to btain their tenacity, the samples of asphalt were melted, poured

[1] *Discussion on* "The Demolition of Waterloo Bridge." Journal Inst. .E., vol. 3 (1935–36), pp. 507–8. (October, 1936.)

Mr. Lloyd. into briquette moulds, and then tested at different temperatures the results plotted ; the resulting graphs clearly indicated the qua of the asphalt. The test was quick, and could be applied to mate before use or when taken out from the pavement for experimen

The slight tilting of the piers, mentioned on p. 411, might perh be due to the action of the tides. The interval from high wate low water at spring tides was about 8 hours 16 minutes, whilst t from low water to high water was 4 hours 10 minutes ; at neap ti however, the interval from high water to low water was 7 h 8 minutes, and from low water to high water 5 hours 27 minu The neap-tide curve was not very different from a sine curve, the spring-tide curve was very unsymmetrical, the rise being m steeper than the fall. The average rise per hour was 4·56 feet springs but only 2·36 feet at neaps, with upland flows of fr 6,000 to 7,000 million gallons per day. That might bring vary pressures to bear on the piers, which might be sufficient to cause very slight tilt which had been referred to.

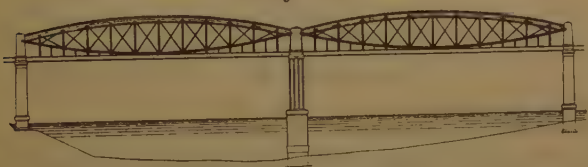
Mr. Gribble. Mr. Conrad Gribble remarked that there was no doubt that considering a bridge over the Thames in the position of Chel bridge, æsthetic considerations, if not paramount, were of the v greatest importance. The Authors had referred to only one alter tive form of construction that might have been adopted, namely four-arch bridge, and they stated that owing to the proximity of railway-bridge that type was not suitable. In considering whet the bridge should be of the self-anchored type or of the ordinary ty of suspension-bridge, however, he took it that the nature of anchorages was really by far the most important consideration enabling the engineers to come to a decision. If such a bri had been erected in a rocky gorge, where there might be an excelle anchorage at a comparatively small cost, the factors affecting t design would have been different ; but in the present case t Authors said quite clearly that an anchorage would be very expensi and it seemed to him, in answering Dr. Anderson's query, that definite rule could be laid down for general cases, because each c would depend on the complication and expense of anchorages. other situations any comparison, he thought, would be not o between a self-anchored suspension-bridge and a normal suspensio bridge, but also between a suspension-bridge and other types bridge. A cantilever-bridge could possibly be built of appro mately the same outline as the suspension-bridge and having central suspended span. That would probably have the very gre disadvantage, however, from an æsthetic point of view, of havi inclined web-members, which, at any rate to his mind, were far l sightly and beautiful than the vertical suspended bars of t

ıspension-bridge. It was not possible, however, to lay down any eneral rules as to the relative economy of the various types of ridge without relation to the foundations, and particularly to the nchorages. Mr. Gribble.

He noticed that the Krupp steel piling used in the cofferdams had s joints on what might be called the tension- and compression- anges of the members. The cofferdam had been extraordinarily ry, and he wondered whether that type of piling had any advantages ı that respect over types where the joints were on the web-members.

Dr. W. L. Lowe-Brown observed that Dr. Anderson had sup- orted the Authors in his suggestion that there were no previous xamples of the self-anchored suspension-bridge in Great Britain. ınother speaker had already mentioned the Albert bridge nearby. Ie would like to draw attention to the Royal Albert bridge at altash with two 455-foot spans (opened in 1859), which was in- lisputably of the self-anchored type (*Fig. 28*). It might be said Dr. Lowe-Brown.

Fig. 28.



hat that bridge was not a self-anchored suspension-bridge in the ense intended by the Authors; in his opinion, however, it was uch a bridge, although the stiffening girders had been placed at he tops of the towers instead of at traffic-level.

Mr. P. W. E. Holloway said that he would like to place on record he names of some of the more prominent sub-contractors who were ıot mentioned in Appendix I to the Paper: Messrs. W. Shaw & Company, Ltd., supplied all the steel castings, and Messrs. Head, Wrightson & Company, Ltd., all the forged steel; Messrs. Cooper, Wettern & Company, Ltd., supplied the Cornish granite for piers ınd abutments, and the Aberdeen granite for the parapets. They ıad played an important part in the successful completion of the work, and had helped to bring about the opening of the bridge for traffic 5 months before the contract date. Mr. P. W. E. Holloway.

He would also like to congratulate the Authors on having been able to present their Paper for discussion so soon after the completion of the bridge.

Mr. J. S. Wilson.

Mr. J. S. Wilson, referring to the question of the pre-stressing the ropes, wished to draw attention to Mr. Banks's Paper on Island of Orleans suspension-bridge, in the Appendix[1] to wh would be found particulars of the tests on the pre-stressed rope that bridge, showing that reeling and unreeling had no apprecia effect on the modulus of elasticity of the pre-stressed ropes.

Another point of interest was the question of settlement. He always understood that the amount of settlement depended v largely on the condition of the surface of the clay when the first of concrete was put on it. In many cases great care was taken w regard to the final surface, but if it were trodden upon by the wo men and softened, the likelihood of settlement would be far grea than if it were protected with either concrete or bricks, or in so other way, immediately after the last layer of clay was taken off. would be of interest to know the exact process adopted at Chel bridge.

He wished to add his contribution in praise of the appearance the bridge. The structure had been so well proportioned that direct appeal was made to the sense of fitness and strength, and t entire absence of anything added for the sake of appearance wa characteristic which he hoped would be taken as an example future bridge design.

Mr. Booth.

Mr. H. C. Booth remarked that in 1895 and in 1897 he had h the ropes which formed the spokes of the great wheels at Blackpo and Paris pre-stressed, and he could say definitely that with tho ropes the pre-stressing had been effective, although the ropes had be reeled and unreeled afterwards. That was in agreement with t previous speaker's remarks on the subject.

One very interesting and fundamental point about all suspensi bridges, and one which affected the whole economy of constructi was the deflexion. The first thing to be settled, in his opinion, w what deflexion or slope was to be allowed on the bridge to suit t class of traffic and the conditions to be met. It was generally in possible to get any specific information on that point. He h recently had some communication with Mr. D. B. Steinman America, and Mr. Steinman had agreed entirely that that was fundamental consideration in the design of all suspension-bridges. the bridge were made a little stiffer, it might greatly increase the co

Mr. Nicholson.

Mr. A. C. Nicholson said that he would like to answer one or t of the questions about the way in which the cables behaved aft being pre-stressed. In the first place, it should be mentioned th on being re-reeled there was a certain apparent loss in the effect

[1] Journal Inst. C.E., vol. 3 (1935–36), pp. 420 and 430. (October, 1936

e-stressing; tests had, however, subsequently been made upon Mr. Nicholson. e cables after reeling and unreeling, and it was found that the ıole of the effect of the pre-stressing had returned. It was evident, erefore, that the effects of pre-stressing were not lost when the bles were erected on the bridge. It might be added that there ıs a great deal of advantage in the system in regard to the marking the cables; the cables were naturally shorter when they were in eir normal unloaded state, and when they were pre-stressed it was ssible accurately to mark the various points to which attachments ere to be made when the cables were on the towers.

Mr. T. H. WEBSTER said that he had seen estimates for the cost of Mr. Webster. aterloo bridge which placed that cost at £1,300,000; also the cost a new bridge at Charing Cross had been given as £9,000,000, but om the Paper it would appear that the actual cost of the new bridge ross the Thames at Chelsea was only about £300,000, which was out one-thirtieth of the cost of the proposed Charing Cross bridge. was striking to compare the cost of £300,000 for a suspension-idge across the Thames with the figure of £1,300,000 for an arch idge, such as at Waterloo.

*** Sir GEORGE HUMPHREYS, Past-President, observed that in Sir George Humphreys. s last phases the old bridge had been under his charge, and it had en his responsibility to arrange for its maintenance. The orna-ental parts of the cast-iron structures of the towers had been a urce of trouble; pieces frequently dropped off, and the suscep-bilities of artistic Chelsea were very disturbed when what were esignedly temporary expedients were resorted to in order to replace slodged portions. The state of the carriageway became so feeble at he advised the removal of the old bridge and the construction a new one just prior to his retirement from the post of Chief ngineer to the London County Council. His recommendation had en adopted; the new bridge formed an important link in the ondon traffic-routes, and, contrary to what had obtained in the ast, could carry motor-bus traffic.

The question had been discussed of the relative merits of founding idge-piers in the Thames by means of compressed-air or free-air ethods, and the examples of the latter method as tried at the new helsea bridge and even more recently at the new Wandsworth idge, were cited as examples of the saving that might be effected y adopting the free-air method. If for any particular bridge ternative Bills of Quantities were prepared for founding in com-essed air or in free air, and tenders were asked for on those data, it ould almost certainly be found that tenderers would quote a lower

*** This contribution was submitted in writing.—SEC. INST. C.E.

Sir George Humphreys. price for the latter. Whether the information furnished by tenders would conclusively prove that ultimate economy w result from the adoption of free air was not so certain. The l forms of steel sheet-piles were undoubtedly a great advance on old timber dams, once the only practical type, but the succe use of steel sheet-piling was largely dependent upon the natu the ground through which the foundation-block had to be constru It was apparent from the discussion that the efficacy of the arra ments for keeping the bed of the foundation dry depended la upon the leakage through the sheeting being intercepted by a forced-concrete bottom frame and conducted away into the su by pipes, and upon the surrounding ground, in the case of both bridges cited, being favourable and not exerting undue pressur the sheeting before it could be well strutted. It thus appeared t given favourable ground, steel sheet-piling could perform the neces function, but in his opinion it provided little or no margin for tingencies should the condition of the ground prove unfavourable— example, if poor clay were met with, or if the level of the clay sh prove to be lower than anticipated, owing to an unexpected furro its surface filled with ballast. In either of such events, he was of opinion that the greater adaptability of the compressed-air met to meet such unexpected difficulties might well prove it to be more economical method in the long run. The question was in esting; in any particular case the engineer concerned should m the most thorough examination beforehand of the actual phy conditions to be encountered, and, after balancing the probabili should arrive at an unfettered decision as to the course to adopted.

The Authors. The AUTHORS, in reply, observed that they had carefully stu the application of the self-anchoring type of suspension-bridge Chelsea, but they had not gone further afield in their investigat than had been necessary. Dr. Anderson had suggested that, altho a self-anchored suspension-bridge might be economical for mode spans, at anything over 1,000 feet it might not prove to be cheapest type, and he had asked how much had been saved adopting the self-anchored type of bridge at Chelsea. To giv true figure would mean a great deal of work; in effect, it would necessary to design an ordinary suspension-bridge for purposes comparison. Actually, the Authors had found that the self-ancho type would be cheaper, and also it had certain advantages, such greater stiffness, over the ordinary anchored type of suspens bridge.

With regard to the settlement of the piers, it had been sugges that 1 inch per ton of additional weight per square foot mi

excessive, but they had only taken that figure as a guide, and did t think it was excessive when applied in the manner described the Paper. The pier had already been constructed, and as the perstructure would increase the loading by about 1 ton per square ot, 1 inch had been allowed for further settlement. Actually, e pier had first settled 1 inch, and had then slowly settled a further inch, probably due to delayed settlement of the first portion. If ything, however, that movement indicated that an allowance of inch per ton per square foot for the added loads, such as those of e superstructure, was not enough. They did not think it would safe to apply the figure of 1 inch per ton per square foot to bridges nerally; the condition of the clay varied, and such an assumption uld be regarded only as a very rough-and-ready method of dealing th the matter. The Authors.

The question raised by Dr. Anderson in regard to the pre-stressing the cables had been answered by subsequent speakers, who ated that the pre-stressing was effective. The great benefit of e-stressing was that the ropes could be marked to the correct ngths; the ropes were all of different lengths (since they traversed ghtly different paths), the maximum variation being nearly 12 ches, and the marking had to be right to a small fraction of an ch. Each rope had been stressed to its working load and had then en marked off, so that when it arrived on the site the exact spot at hich each clip was to be attached, and the points where it was to oss each tower, were indicated. The pre-stressing could not have en carried out on the site at Chelsea, as had been done in Germany.

Although Mr. Firth had raised the question of whether or not e new Chelsea bridge was the first self-anchored suspension-bridge England, the Authors still thought that it was the first. Mr. irth had referred to the Albert bridge; that bridge, however, was ot a suspension-bridge, but was a composite bridge. It had radial ds which gave it a cantilever form, and combined with that it had spension-chains. It was, however, in some respects similar to a lf-anchored suspension-bridge in that it had vertical shore chorages. Dr. Lowe-Brown had instanced the Royal Albert idge at Saltash as an example of a self-anchored type of bridge. was probable that no engineering design could be said to be tirely new, but there was very little similarity between the Royal lbert and Chelsea bridges.

Mr. Firth had also mentioned that in the case of Wandsworth idge the use of either caissons or open cofferdams for the piers had en considered, and that in view of the satisfactory results obtained at helsea it had been decided to leave that matter optional when inviting nders, with the result that the open cofferdam had proved cheaper

The Authors. and there had been a saving of £12,000. The relative meri caissons and open cofferdams had often been discussed, and l Sir George Humphreys and Mr. Peirson Frank had been inter in the matter. The Authors themselves believed in doing wo the open if it were at all possible.

Mr. Firth had asked for further information with regard t deflexion of the steel piling, but the Authors had not much information than was given in the Paper. Such measurem like those of the temperature of the concrete, were incide they could easily be taken while the work was proceeding, bu they had not been of paramount importance only such recor were convenient had been taken.

Mr. W. Storey Wilson had given some interesting particulars the contractors' point of view, and the Authors wished to take opportunity of congratulating the contractors on the excellen their work, which had been carried out in very happy conditi everything had gone very smoothly, and, although the work was quickly, there was no sacrifice of quality. Mr. Wilson did not a that there was an economy to the contractors in the design of cofferdam, but he had thanked the engineers for the opportu of adopting that system; in other words, he thanked the engin for giving him the opportunity of losing money! The Authors not been very enthusiastic over the scheme at first, at least so as the reinforced-concrete frame was concerned, because w excavating in that way it was desirable to be able to take up thrust as excavation proceeded, and it was some time befor reinforced-concrete frame would take its load. In many soils t would not adopt that system or that type of frame, but would u timber frame and would take up the thrust before the soil out would be able to push in the sides of the trench. In the case question, however, conditions had been so favourable and the had been so good that the method adopted had made an excel job, and they were sorry if the contractors had really lost mo on it.

Mr. Lloyd had mentioned the tilt of the piers, and had given interesting outline of the tide-effects. Some tilt was to be expect a very large mass was settling, and it was not to be expected tha would go down absolutely vertically. The amount of tilt, as measu at the tower bases, was only about ¼ inch at the most.

Mr. Gribble, in dealing with the question of self-anchoring, answered Dr. Anderson. The new Chelsea bridge had to carry v heavy modern loadings, and anchorages would have had to be v good. The cantilever type with a suspended span was quite possi but Mr. Gribble did not think the appearance would be very g

Chelsea. Mr. Gribble had also referred to the cofferdam piles and the question of whether or not it was an advantage to have the nt on the flanges instead of in the web at the neutral axis. e Authors did not know what the effect was on watertightness, t the piles adopted were very satisfactory from the point of w of both stiffness and watertightness, and on most jobs they uld prefer to have the locks of the piles away from the neutral is. The Authors.

Mr. P. W. E. Holloway had observed that certain sub-contractors d not been mentioned in the Paper; if names had been omitted ich should have been mentioned, they were sorry.

In referring to the foundations, Mr. J. S. Wilson had mentioned e importance of a good surface for the first lift of concrete. Firstly, equate drainage was essential in order to keep the bottom as y as possible; secondly, the final surface should be exposed as e as possible, and the placing of concrete should commence as on as the last spit had been taken off. The exact condition of the rface as regarded wetness did not matter so much if the concrete re placed as soon as the final surface was exposed.

Mr. Webster had referred to the suspension-bridge at Chelsea sting £300,000 and to Waterloo bridge costing £1,300,000 (although tually the comparable contract figures were £310,737 and £647,624), ther suggesting that if the London County Council were to consult e same engineers they would get a much cheaper bridge at Waterloo. so happened that the same engineers were in charge of both jobs, d the difference in price was due to local conditions requiring fferent types of bridge.

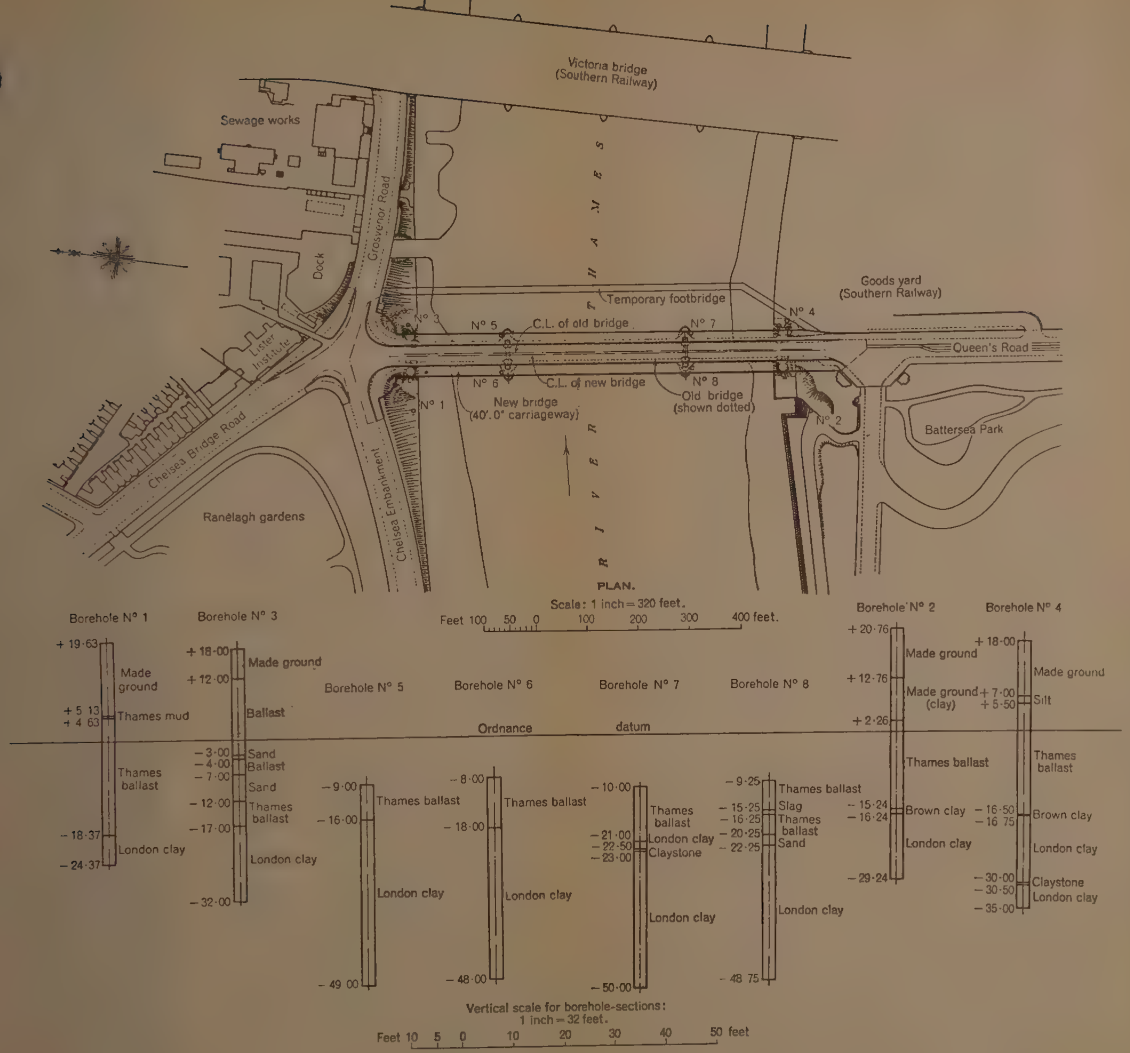
Sir George Humphreys was of course fully aware of the advantages caissons over the older types of cofferdams, but, as he had implied, e advance in recent years in steel sheet-piling and pile-driving d changed the relative merits of caissons and open trenches for e construction of bridge-foundations at moderate depths. As r George had pointed out, one of the advantages of caissons over y type of open trench was that with caissons it was easier to go eper than the designed founding depth if the bottoming proved favourable. In rivers such as those of India, Burma, and Africa, here uncertainties were much greater than on the Thames, open enches were seldom used. The caisson principle was adopted by nking monoliths, usually by open grabbing, with provision for mpressed air if found necessary.

In the case of Chelsea bridge the general formation was well own and the exact local formation had been investigated by rings at an early stage, and open trenches had been decided upon. he conditions for Lambeth bridge were very similar, but there

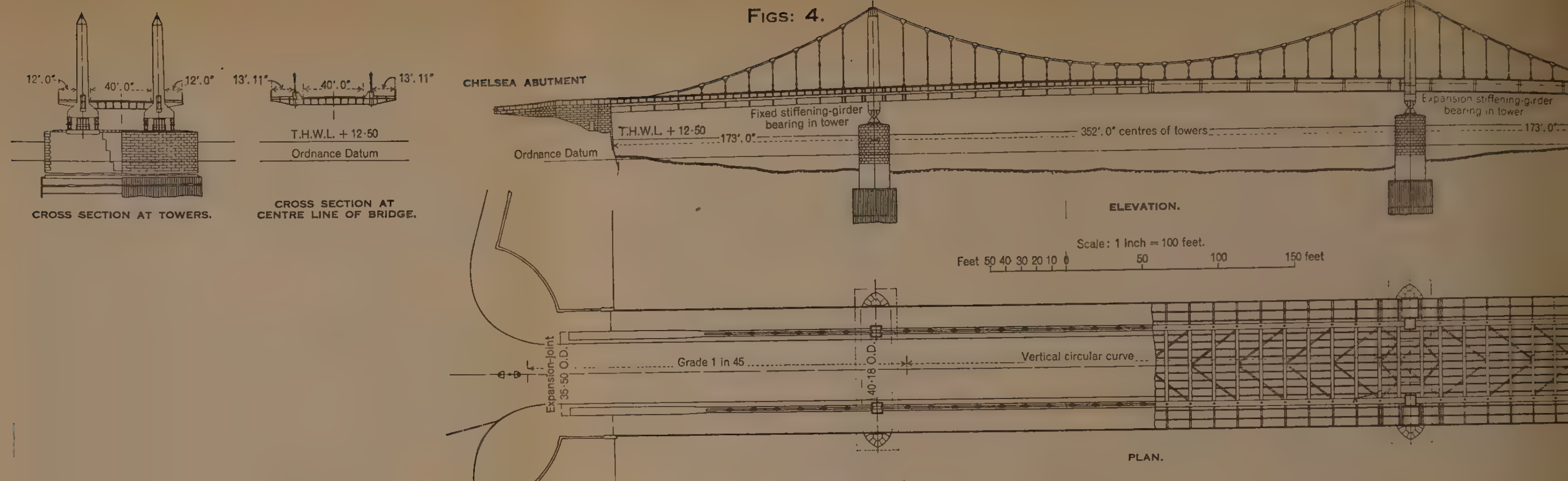
The Authors. caissons had been adopted. The Authors considered caissons ' not likely to be used for the foundations of any future bridges al Tower bridge.

*** The Correspondence on the foregoing Paper will be publis in the Institution Journal for October, 1938.—Sec. Inst. C.E.

Figs: 3.

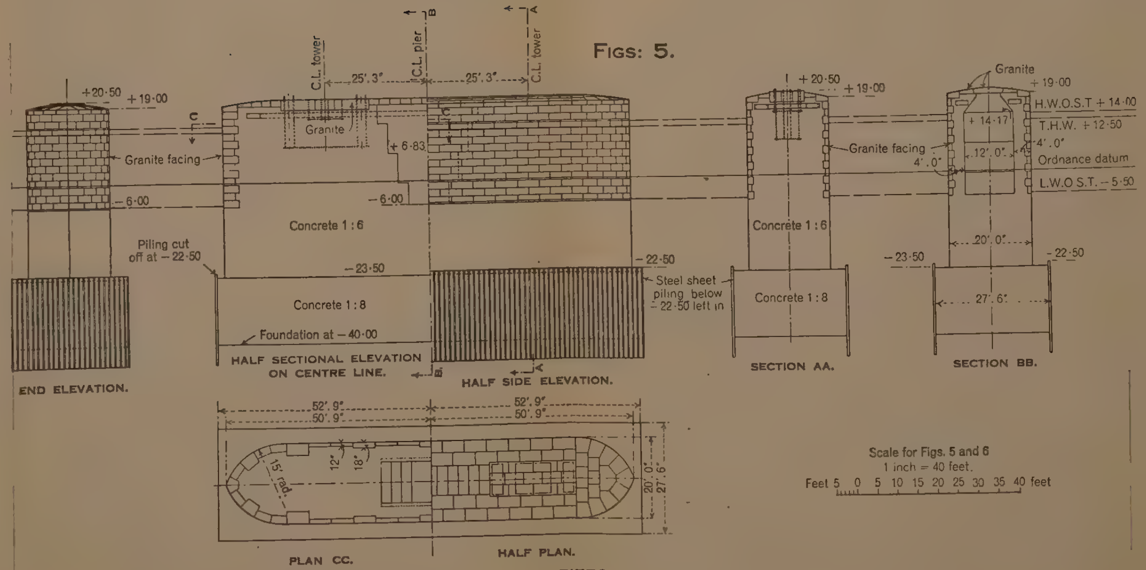


SITE-PLAN AND BOREHOLES.

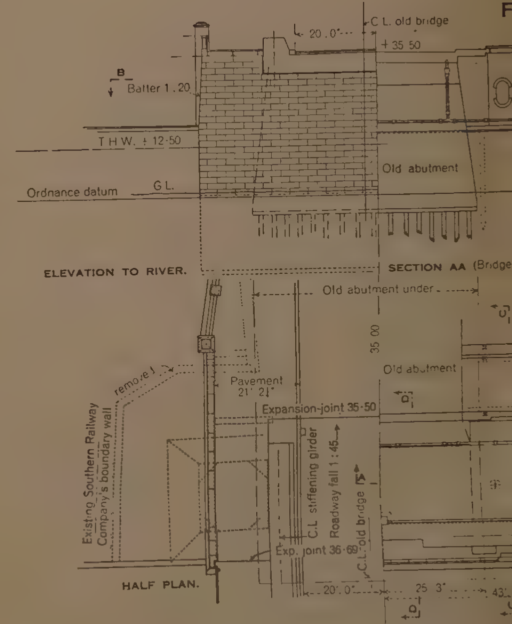


GENERAL ARRANGEMENT.

Figs: 5.

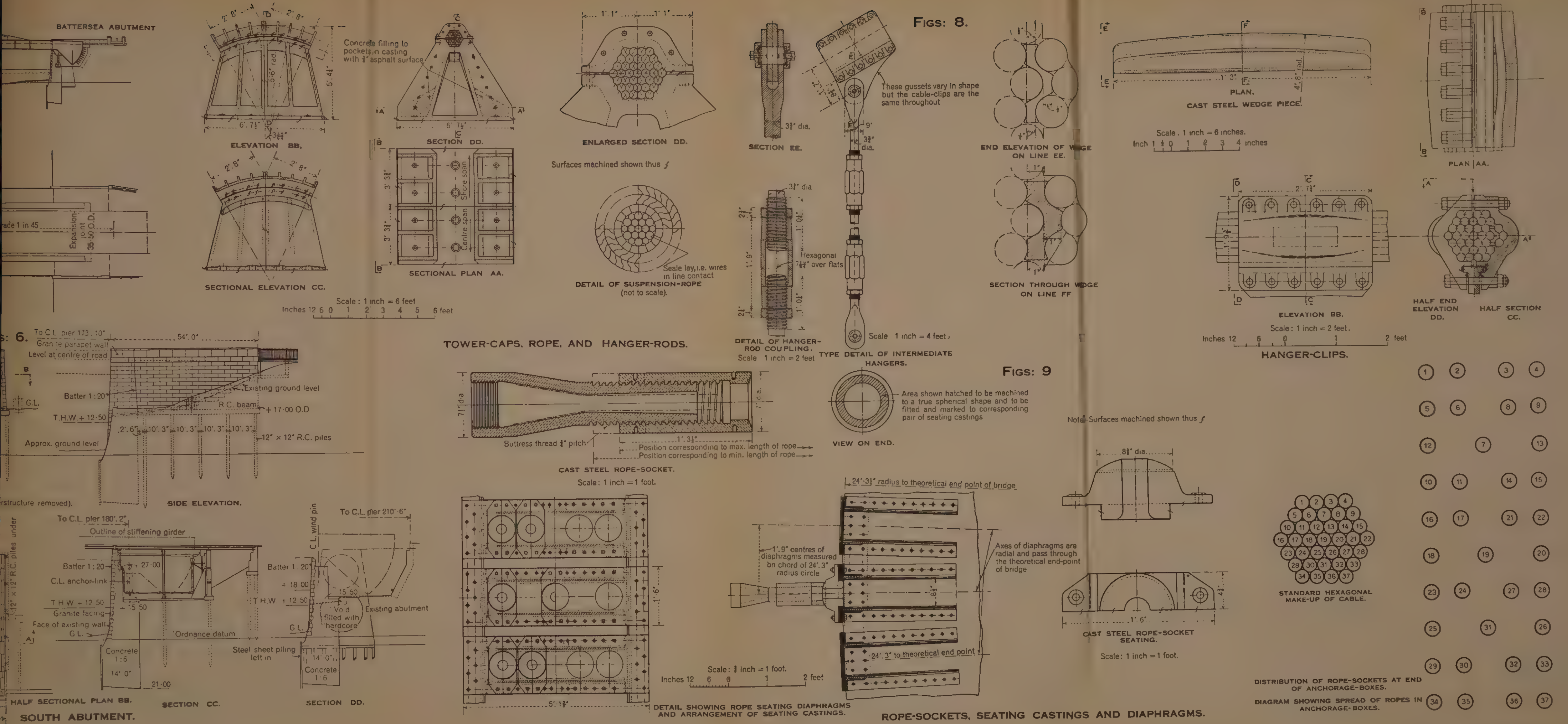


PIERS.



EA BRIDGE.

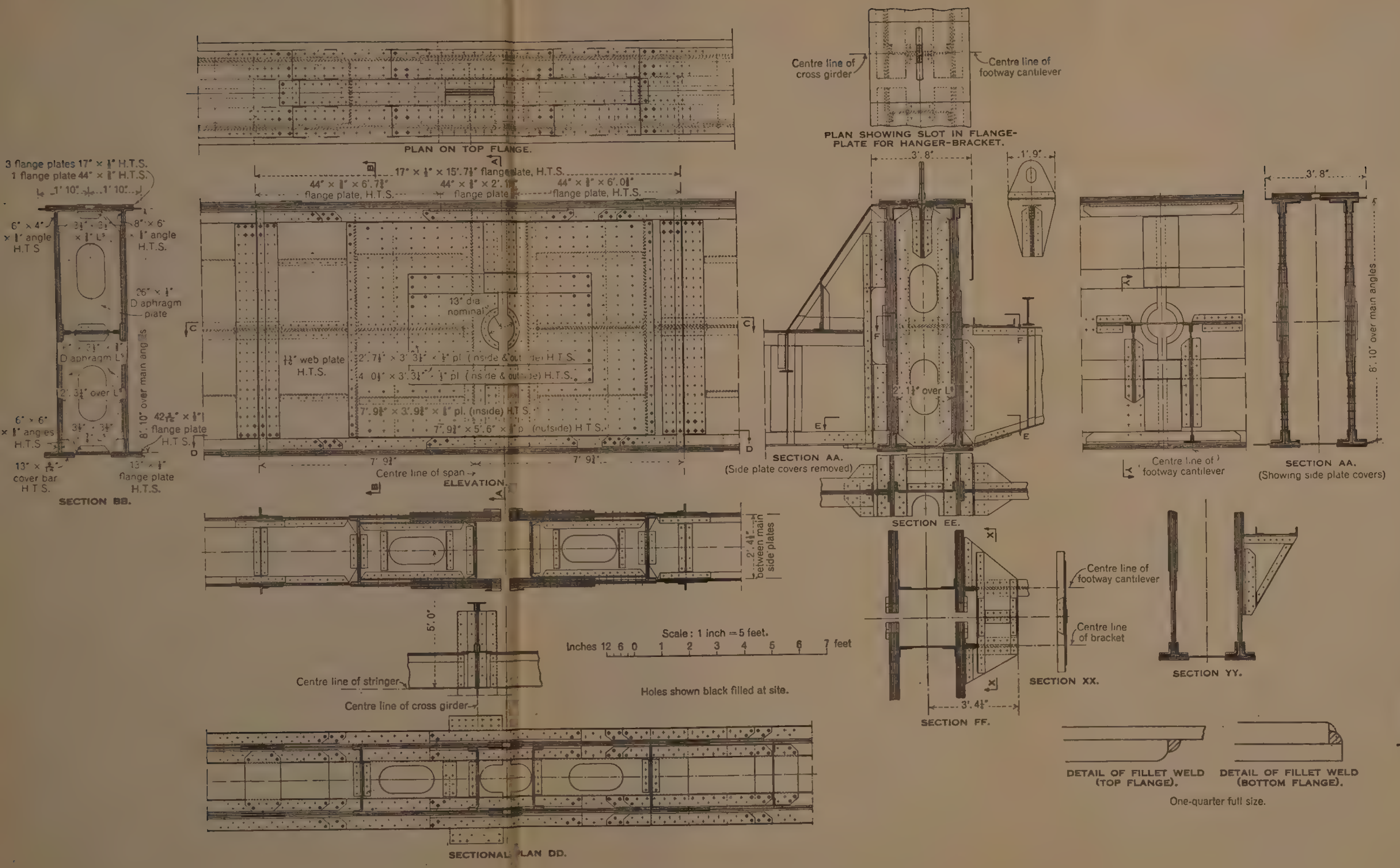
PLATE 1.
THE RECONSTRUCTION OF CHELSEA BRIDGE.



E. J. BUCKTON and H. J. FEREDAY.

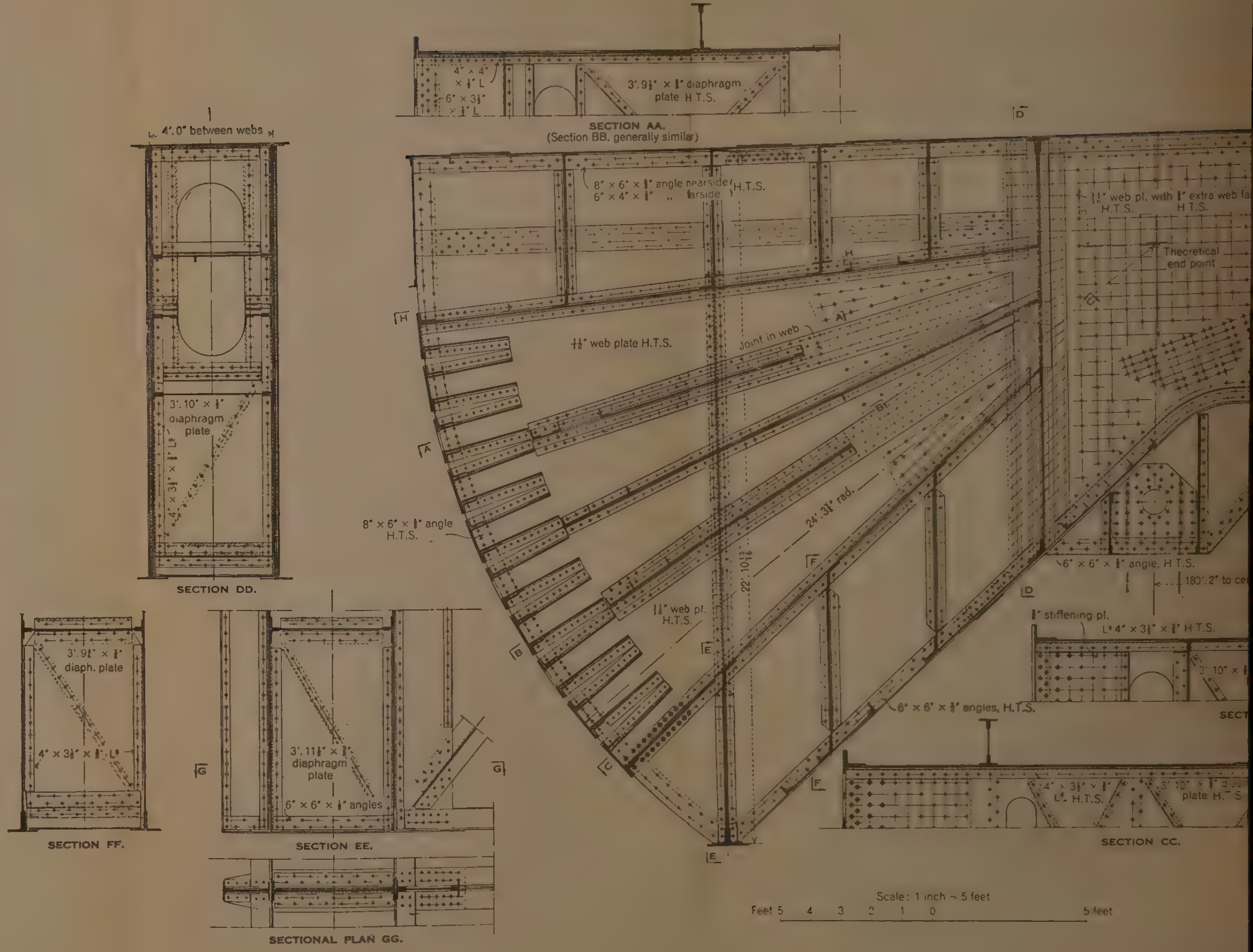
THE RECONSTRUCTION OF CHELSEA BRIDGE.

FIGS: 12.



STIFFENING GIRDER; CENTRE HINGE.

FIGS: 13.

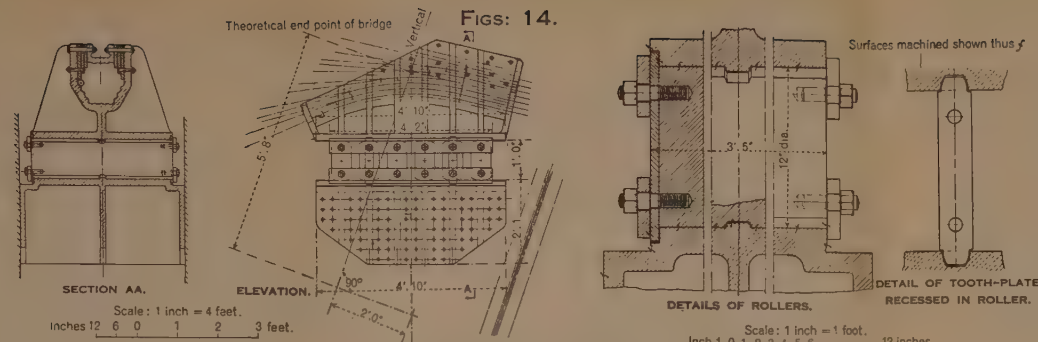


STIFFENING GIRDER: ANCHORAGE-SECTION.

William Clowes & Sons, Limited: London.

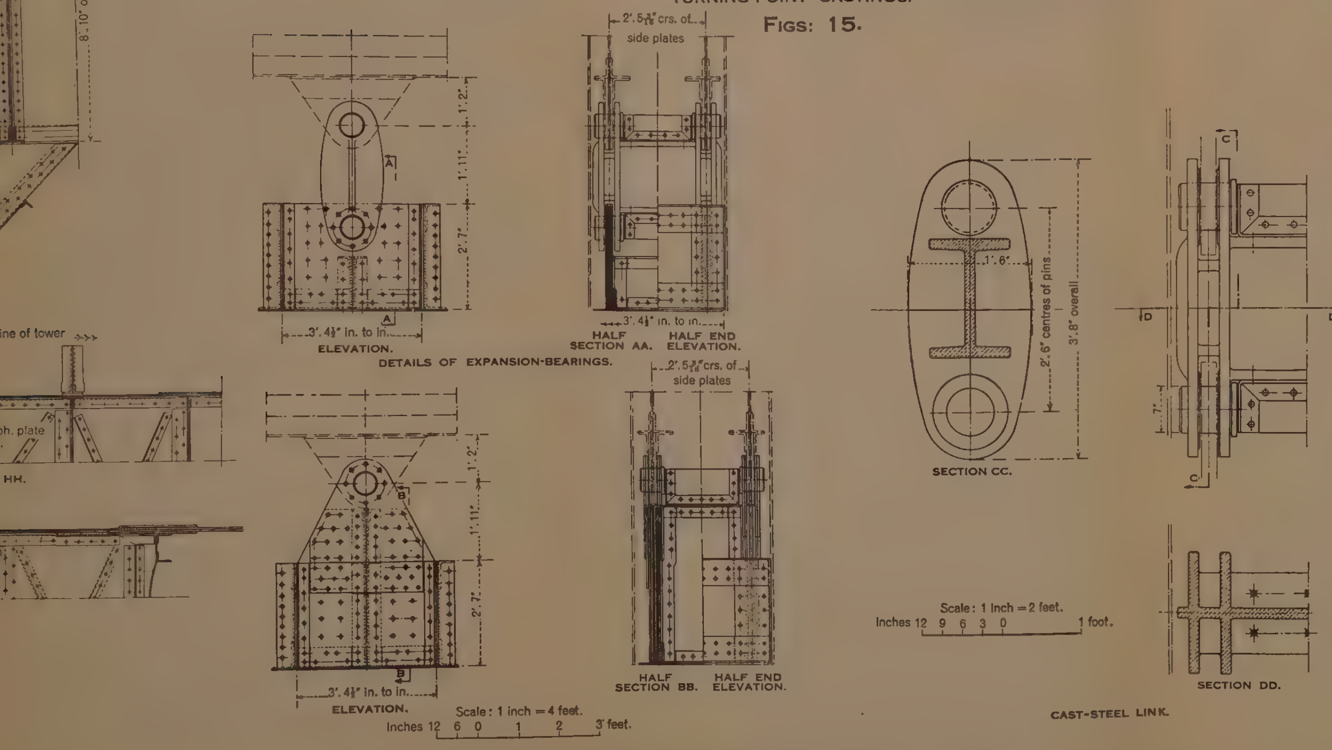
PLATE 2.
THE RECONSTRUCTION OF CHELSEA BRIDGE.

Figs: 14.



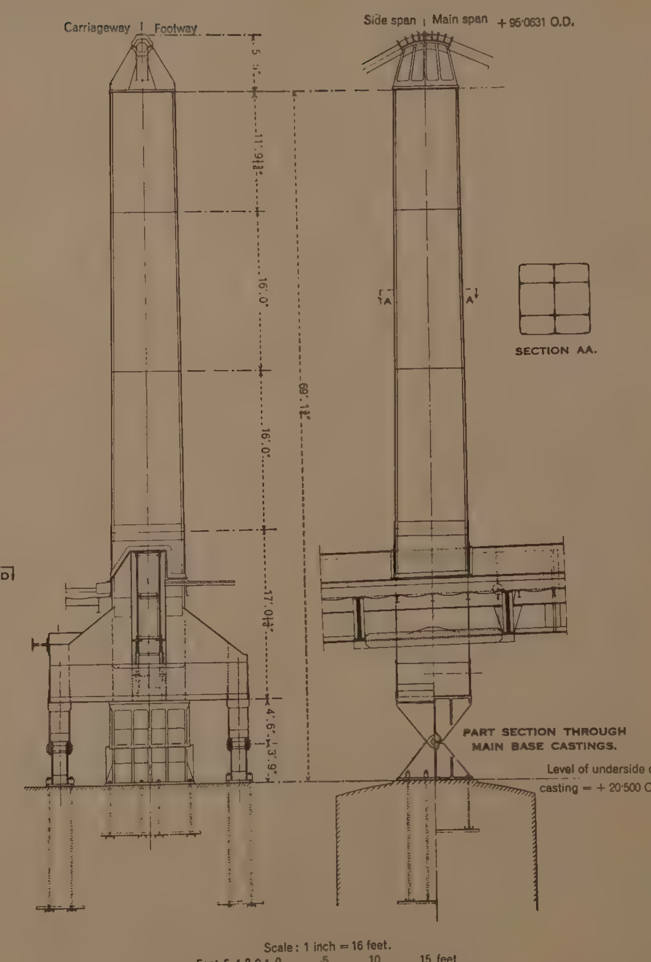
TURNING-POINT CASTINGS.

Figs: 15.



STIFFENING-GIRDER BEARINGS AT TOWERS.

Figs: 16.



TOWER: GENERAL ARRANGEMENT.

E. J. BUCKTON and H. J. FEREDAY.

Paper No. 5145.

"An Application of the Principle of Superposition to Certain Structural Problems."

By Professor ALFRED JOHN SUTTON PIPPARD, M.B.E., D.Sc., M. Inst. C.E.

(Ordered by the Council to be published with written discussion.) [1]

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Multiple loads on a structure	463

INTRODUCTION.

HOOKE'S law states that if a "springy body" be loaded the resulting displacements are proportional to the forces which produce them,[2] and experience shows that many engineering structures obey this law.

It is a direct corollary that the effect produced upon such a structure by a number of loads which act simultaneously is the same as the algebraic sum of the effects produced if the loads are assumed to act separately. This is the Principle of Superposition, which is well known as a theorem although its practical application to the analysis of structures is sometimes not fully appreciated. The object of the present Paper is to show how, by its help, comparatively simple solutions can often be obtained of problems which are somewhat intractable by more direct methods of attack. The method described is applicable only to structures which are symmetrical about a centre-line.

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[1] Correspondence on this Paper can be accepted until the 15th April, 1938, and will be published in the Institution Journal for October, 1938.—SEC. INST. C.E.

[2] R. V. Southwell, "Theory of Elasticity," p. 4. London, 1936.

Principle of Theorem.

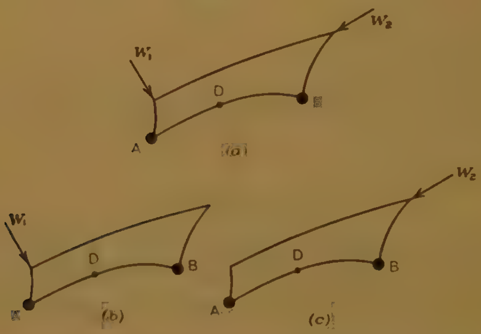
To illustrate the principle, let *Figs. 1* (*a*) represent any body which obeys Hooke's law, carrying loads W_1 and W_2 as shown and supported at the points A and B.

The load-system is the sum of the two systems shown in (*b*) and (*c*), and the stress at any point, such as D, which is produced by the load-system (*a*) is the algebraic sum of the stresses produced by the load systems (*b*) and (*c*), whilst the displacement at any point due to (*a*) is the algebraic sum of the displacements due to (*b*) and (*c*). For the sake of brevity this statement will be expressed in the form :

$$(a)=(b)+(c).$$

It is important to realize that the structures of (*a*), (*b*) and (*c*) are identical ; no modification must be made. This warning is necessary since in some text-books a so-called method of superposition is given

Figs. 1.



in which a redundant structure is divided into two simply-stiff frames and the loading is applied half to one and half to the other. A solution of the stress-distribution obtained in this way is only approximate ; it may in some cases be a good approximation but it cannot, in general, be exact. This method must not be confused with that now to be described, which furnishes exact solutions.

The examples which follow are in no way exhaustive but are chosen solely to illustrate one way in which the principle can be applied to the solution of typical problems. In some of the cases illustrated other methods may be preferable ; the particular method used will always be, to some extent, a matter of personal inclination.

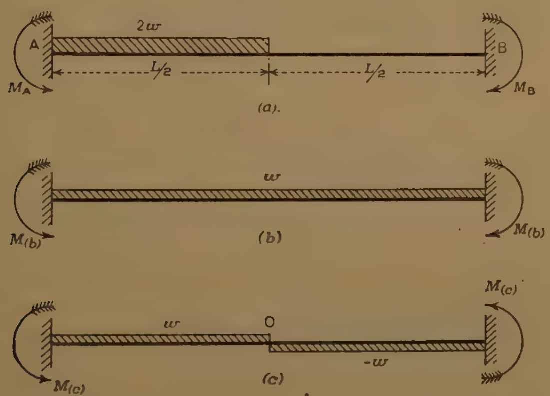
Application to Encastré Beams.

The usual treatment of the encastré beam is based upon the facts that the area of the free-bending-moment diagram is equal to the

area of the fixing-moment diagram, and that the moments of these areas about any point are also equal. If the free-bending-moment diagram is of a simple form this method is quite straightforward, but if it has to be plotted and a planimeter used for the determination of its area and moment it becomes laborious.

Suppose, for example, that the encastré beam shown in *Figs. 2* (*a*) carries a uniform load of intensity $2w$ over one-half of the span, and that it is required to determine the values of the end fixing moments M_A and M_B. The standard method referred to above necessitates either the use of a planimeter or the mathematical integration of the free-bending-moment diagram, and is unnecessarily

Figs. 2.



lengthy. The load-system (*a*) can, however, be replaced by the two systems shown in (*b*) and (*c*) respectively in *Figs. 2* ; (*b*) consists of a uniform load of intensity w acting on the whole span, while (*c*) consists of a downward load of intensity w over one-half of the span and an upward load of intensity w over the other half.

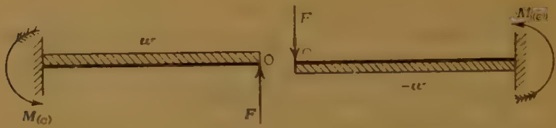
Then $$(a)=(b)+(c).$$

Now, b is a standard case of loading given in all text-books and the end fixing moment is

$$M_{(b)}=\frac{wL^2}{12}.$$

It is clear from the skew-symmetry of the arrangement that in the case of loading shown at (*c*) there is neither bending moment nor deflexion at O, the centre of the beam. There is, however a shearing force F, and the two halves of the beam are in equilibrium under the actions shown in *Figs. 3.*

Figs. 3.



Considering the left-hand half, the upward deflexion of O due to F equals the downward deflexion of O due to w.

That is,
$$\frac{FL^3}{24EI} = \frac{wL^4}{128EI},$$

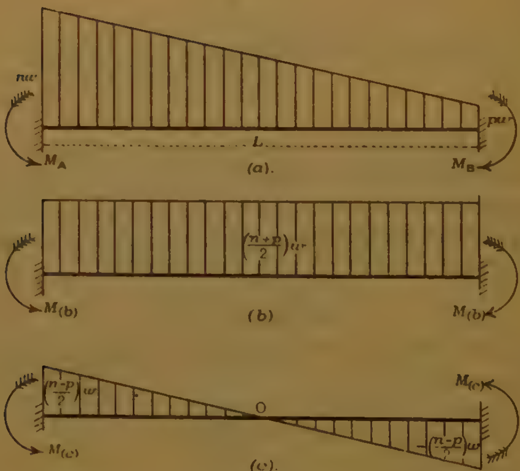
whence
$$F = \frac{3}{16}wL,$$

and
$$M_{(c)} = \frac{wL^2}{8} - \frac{3wL^2}{32} = \frac{wL^2}{32}.$$

Hence
$$M_A = M_{(b)} + M_{(c)} = \frac{wL^2}{12} + \frac{wL^2}{32} = \frac{11wL^2}{96},$$

and
$$M_B = M_{(b)} - M_{(c)} = \frac{wL^2}{12} - \frac{wL^2}{32} = \frac{5wL^2}{96}.$$

Figs. 4.



As a second example, consider the encastré beam loaded as shown in *Figs. 4* (*a*). The load in this case varies linearly from an intensity of nw at one end to one of pw at the other, and can be divided into two systems as shown in (*b*) and (*c*). The first of these is a load of

uniform intensity $\left(\dfrac{n+p}{2}\right)$ w over the whole span, and the second is a load varying linearly from $\left(\dfrac{n-p}{2}\right)$ w at one end to $-\left(\dfrac{n-p}{2}\right)w$ at the other.

As before, (b) is a standard case and the end fixing moment is

$$M_{(b)} = \frac{(n+p)wL^2.}{24}$$

The loading shown at (c) produces neither a bending moment nor a deflexion at O but, as in the first example, there is a shearing force F.

The left-hand half of this beam may be considered to be a cantilever loaded as shown in *Fig. 5*. The deflexion at O is, by the first

Fig. 5.

theorem of Castigliano, given by $\dfrac{dU}{dF}$ for any value of F, and since this deflexion is zero F can be determined from the equation $\dfrac{dU}{dF} = 0$;

that is, $$\int_0^{L/2} M_x \frac{dM_x}{dF}\,dx = 0.$$

Then, putting $$\frac{n-p}{2} = k,$$

$$M_x = \frac{kwx^3}{3L} - Fx\,; \quad \frac{dM_x}{dF} = -x,$$

and $$\int_0^{L/2} \left(\frac{kwx^4}{3L} - Fx^2\right)dx = 0.$$

Upon integration and reduction this gives

$$F = \frac{kwL}{20},$$

and so $$M_{(c)} = \frac{kwL^2}{24} - \frac{kwL^2}{40} = \frac{kwL^2}{60}.$$

Then $$M_A = M_{(b)} + M_{(c)} = \left(\frac{n+p}{24} + \frac{n-p}{120}\right) wL^2,$$
$$= \frac{wL^2}{60}(3n+2p),$$

and $$M_B = M_{(b)} - M_{(c)} = \left(\frac{n+p}{24} - \frac{n-p}{120}\right) wL^2,$$
$$= \frac{wL^2}{60}(2n+3p).$$

It may be noted that the central deflexion of the beam under the load system (a) is the sum of the central deflexions due to (b) and (c), and since that due to (c) is zero the central deflexion is

$$\frac{1}{384}EI\left(\frac{n+p}{2}\right) wL^4 = \frac{(n+p)wL^4}{768EI}.$$

Application to the Elastic Arch.

An excellent example of the utility of this method is afforded by the case of the elastic arch-rib encastré at the ends. Portals may be treated in an exactly similar manner. In this type of structure there are three redundant elements which may conveniently be taken to be the bending moment, the thrust, and the vertical reaction at one end.

By using strain-energy methods three simultaneous equations can be obtained which, on solution, give the values of these reactions. The equations are, however, involved, and an explicit solution in general terms is extremely cumbersome. If the Principle of Superposition is used, however, the work is simplified very considerably and a general solution is readily found.

A segmental arch of the type under discussion is shown in *Figs.* 6. It is encastré at A and B and carries a load $2W$ at an angle ψ from the axis of symmetry OC. The reactions at A are V_A, H and M_A, and at B they are V_B, H and M_B.

The load-system is split up as shown in *Figs.* 6 (b) and (c), so that, as before,

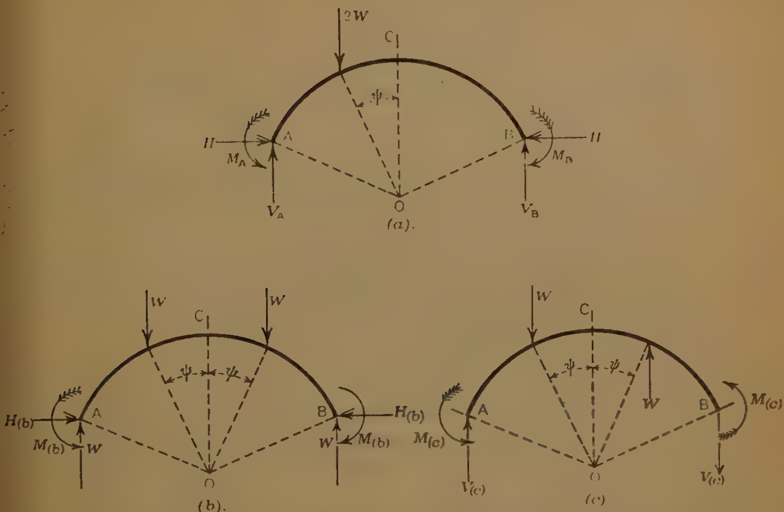
$$(a) = (b) + (c).$$

The reactions at A and B under the symmetrical system (b) consist of equal vertical reactions W, a thrust H_b and equal fixing moments M_b.

Under the skew-symmetrical system (c) there is no thrust, the vertical reactions are equal and opposite, of magnitude V_c, and the fixing moments are equal and opposite, of magnitude M_c.

Since in case (b) the loading is symmetrical it is only necessary to

Figs. 6.



consider one half of the arch, and with the notation shown in *Figs.* 7 (*b*) the bending moment at any point X will be,

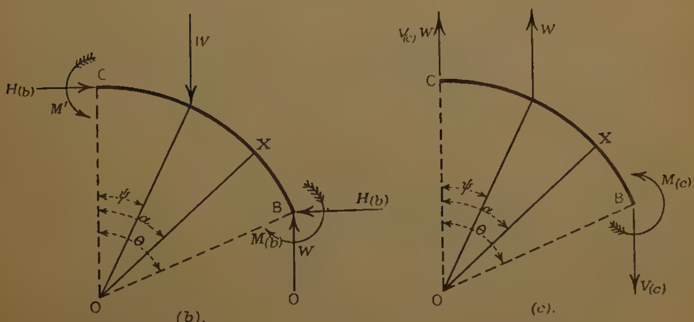
$$M_x = M' - H_bR(1 - \cos\alpha) + WR(\sin\alpha - \sin\psi),$$

where M' is the bending moment at the crown and the last term only occurs when α is greater than ψ. The conditions to be satisfied are

$$\frac{\partial U}{\partial M'} = 0, \quad \text{and} \quad \frac{\partial U}{\partial H_b} = 0.$$

Now $$\frac{\partial M_x}{\partial M'} = 1, \quad \text{and} \quad \frac{\partial M_x}{\partial H_b} = -\mathrm{R}(1 - \cos\alpha),$$

Figs. 7.



and the equations are therefore

$$\int M_x d\alpha = 0$$

and $$\int M_x(1 - \cos\alpha)d\alpha = 0.$$

But $\int M_x(1 - \cos\alpha)d\alpha = \int M_x d\alpha - \int M_x \cos\alpha d\alpha = 0 - \int M_x \cos\alpha d\alpha$. The equations to be solved are therefore

$$\int_0^\theta \{M' - H_b R(1 - \cos\alpha)\}d\alpha + WR\int_\psi^\theta (\sin\alpha - \sin\psi)d\alpha = 0,$$

and

$$\int_0^\theta \{M' - H_b R(1 - \cos\alpha)\}\cos\alpha d\alpha + WR\int_\psi^\theta (\sin\alpha - \sin\psi)\cos\alpha d\alpha = 0.$$

By integration and reduction

$$M'\theta - H_b R(\theta - \sin\theta) + WR\{\cos\psi - \cos\theta - (\theta - \psi)\sin\psi\} = 0,$$

$$4M'\sin\theta - H_b R(4\sin\theta - 2\theta - \sin 2\theta) + WR\{\cos 2\psi - \cos 2\theta - 4\sin\psi(\sin\theta - \sin\psi)\} = 0,$$

and these give

$$H_b = W\frac{\theta(\cos 2\psi + \cos 2\theta - 2) + 4\sin\theta(\cos\psi - \cos\theta + \psi\sin\psi)}{2\theta^2 + \theta\sin 2\theta + 2\cos 2\theta - 2}.$$

By taking moments about B,

$$M_b = M' - H_b R(1 - \cos\theta) + WR(\sin\theta - \sin\psi),$$

whence, substituting for M' in terms of H_b from the first equation,

$$M_b = \frac{R}{\theta}[H_b(\theta\cos\theta - \sin\theta) - W\{(\cos\psi + \psi\sin\psi) - (\cos\theta + \theta\sin\theta)\}].$$

Let the skew symmetrical case shown in *Figs.* 6 (*c*) be now considered. From the symmetry there can be no bending moment at C, and since there is no vertical movement of this point, it is again only necessary to deal with one half, as shown in *Figs.* 7 (*c*).

The bending moment at X is

$$M_x = (V_c - W)R\sin\alpha + WR(\sin\alpha - \sin\psi),$$

the second term only appearing when α is greater than ψ.

The condition to be satisfied is

$$\frac{dU}{dV_c} = 0.$$

Since $\frac{dM_x}{dV_c} = R\sin\alpha$, this condition gives the equation

$$(V_c - W)R\int_0^\theta \sin^2\alpha d\alpha + WR\int_\psi^\theta (\sin^2\alpha - \sin\alpha\sin\psi)d\alpha = 0.$$

Integrating this and reducing gives

$$V_c = \frac{W(2\psi + \sin 2\psi - 4\cos\theta \sin\psi)}{2\theta - \sin 2\theta},$$

$$M_c = \frac{WR\{2\psi \sin\theta - \sin\psi(2\theta + \sin 2\theta) + \sin 2\psi \sin\theta\}}{2\theta - \sin 2\theta}.$$

Superposing the two sets of results, the complete solution is

$$H = W\frac{\theta(\cos 2\psi + \cos 2\theta - 2) + 4\sin\theta(\cos\psi - \cos\theta + \psi\sin\psi)}{2\theta^2 + \theta\sin 2\theta + 2\cos 2\theta - 2}.$$

$$V_A = W\left\{1 + \frac{2\psi + \sin 2\psi - 4\cos\theta\sin\psi}{2\theta - \sin 2\theta}\right\}.$$

$$V_B = W\left\{1 - \frac{2\psi + \sin 2\psi - 4\cos\theta\sin\psi}{2\theta - \sin 2\theta}\right\}.$$

$$M_A = M_b + M_c.$$

$$M_B = M_b - M_c.$$

It will be noticed that, although there are still three redundancies to be evaluated, two in case (*b*) and one in case (*c*), this involves only two simultaneous equations, which occur in case (*b*). If the problem were attacked directly there would be three simultaneous equations to be solved, and in addition, the number of terms would be considerably greater and the whole process very much more involved.

Application to Bow Girders.

The general treatment of the problem of the girder curved in plan has been given by the Author [1] and the resulting equations, while easily soluble in numerical cases, do not readily yield explicit solutions in general terms. By an application of the Principle of Superposition, however, such explicit solutions can be obtained quite readily.[2]

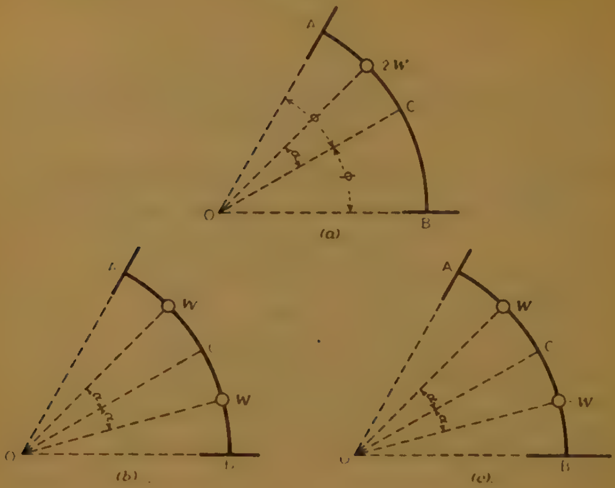
Suppose, for example, a girder in plan form consists of the arc of a circle subtending an angle 2ϕ at the centre, as shown in *Figs. 8* (*a*). A load $2W$ is carried at an angle α from the centre.

This loading can be replaced by two systems as shown in (*b*) and (*c*) in the same figure. The load-system (*b*) consists of two equal

[1] "Strain Energy Methods of Stress Analysis," pp. 41 *et seq.* London, 1928.

[2] The Author is indebted to Miss Letitia Chitty for the calculation of the following example.

Figs. 8.



loads W at $\pm\alpha$ from the centre-line, and the load-system (c) consists of a load W acting downwards at α and an equal load acting upwards at $-\alpha$.

Considering first the load-system (b), it is clear that at the centre of the beam the only resultant action is a bending moment M_{D}. Both the torque T_{D} and the shearing force F_{D} are zero.

In *Figs. 9* (b), if X is any point at θ from the centre of the girder, then, using Macaulay's notation,

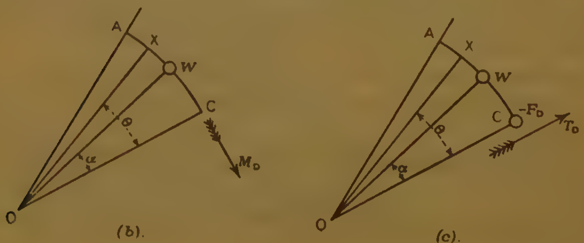
$$M_x = M_{\mathrm{D}} \cos\theta + [WR \sin(\theta - \alpha)]$$

and

$$T_x = -M_{\mathrm{D}} \sin\theta - [WR\{1 - \cos(\theta - \alpha)\}].$$

Then, since $\dfrac{\partial U}{\partial M_{\mathrm{D}}} = 0$, the following equation can be formed for

Figs. 9.



the evaluation of M_D,

$$\frac{1}{EI}\left[\int_0^\phi M_D \cos^2\theta d\theta + \int_\alpha^\phi WR \sin(\theta-\alpha)\cos\theta d\theta\right]$$
$$+\frac{1}{NJ}\left[\int_0^\phi M_D \sin^2\theta d\theta + \int_\alpha^\phi WR\{1-\cos(\theta-\alpha)\}\sin\theta d\theta\right]=0,$$

where EI is the flexural rigidity and NJ is the torsional rigidity of the beam.

Upon integration and reduction this gives the value of M_D.

If M_b and T_b are the values of the fixing-moment and torque respectively at the ends of the beam,

$$M_b = M_D \cos\phi + WR\sin(\phi-\alpha)$$

and
$$T_b = -M_D\sin\phi - WR\{1-\cos(\phi-\alpha)\}.$$

Thus, having found M_D by the solution of one equation, M_b and T_b can be obtained. These are given by

$$\begin{aligned} &M_b[EI(2\phi - \sin 2\phi) + NJ(2\phi + \sin 2\phi)] \\ &\quad = WR[(EI+NJ)\{(\phi-\alpha)\sin(\phi+\alpha) + (\phi+\alpha)\sin(\phi-\alpha)\} \\ &\qquad + EI.4\cos\phi(\cos\phi-\cos\alpha)] \end{aligned}$$

and

$$\begin{aligned} &T_b[EI(2\phi - \sin 2\phi) + NJ(2\phi + \sin 2\phi)] \\ &\quad = -WR[(EI+NJ)\{2\phi - (\phi-\alpha)\cos(\phi+\alpha) \\ &\qquad - (\phi+\alpha)\cos(\phi-\alpha) + 2\sin\phi(\cos\phi-\cos\alpha)\}]. \end{aligned}$$

Now, considering load-system (c), the resultant actions at the centre of the beam consist of a torque T_D and a shearing force F_D; the bending moment is zero.

As before, at α from the centre of the beam in *Figs. 9* (c),

$$M_x = T_D\sin\theta - F_D R\sin\theta + [WR\sin(\theta-\alpha)],$$

and
$$T_x = T_D\cos\theta + F_D R(1-\cos\theta) - [WR\{1-\cos(\theta-\alpha)\}].$$

Here there are two unknown redundant actions, and it is necessary to solve the simultaneous equations derived from

$$\left.\begin{aligned} \frac{\partial U}{\partial T_D} &= 0 \\ \frac{\partial U}{\partial F_D} &= 0 \end{aligned}\right\}.$$

These equations are formed as before, and using the relations

$$M_c = (T_D - F_D R)\sin\phi + WR\sin(\phi-\alpha),$$
$$T_c = (T_D - F_D R)\cos\phi + F_D R - WR\{1-\cos(\phi-\alpha)\},$$

and
$$F_c = W - F_D,$$

where M_c, T_c and F_c are the resultant actions at the end of the beam, it is found that

$$M_c[EI\{\phi(2\phi + \sin 2\phi) - 4\sin^2\phi\} + NJ\{\phi(2\phi - \sin 2\phi)\}]$$
$$= -WR[(EI + NJ)\phi\{(\phi - \alpha)\sin(\phi + \alpha) - (\phi + \alpha)\sin(\phi - \alpha)\}$$
$$+ EI\{4\sin\phi(\phi\sin\alpha - \alpha\sin\phi)\}],$$

$$T_c[EI\{\phi(2\phi + \sin 2\phi) - 4\sin^2\phi\} + NJ\{\phi(2\phi - \sin 2\phi)\}]$$
$$= -WR(EI + NJ)[2\phi\alpha + \phi\{(\phi - \alpha)\cos(\phi + \alpha)$$
$$- (\phi + \alpha)\cos(\phi - \alpha)\} - 2\alpha\sin\phi(\cos\phi - \cos\alpha)],$$

and

$$F_c[EI\{\phi(2\phi + \sin 2\phi) - 4\sin^2\phi\} + NJ\{\phi(2\phi - \sin 2\phi)\}]$$
$$= W[(EI + NJ)\{2\phi\alpha - (\phi - \alpha)\sin(\phi + \alpha) + (\phi + \alpha)\sin(\phi - \alpha)\}$$
$$+ EI(\alpha\sin 2\phi - 4\sin\phi\sin\alpha) - NJ\alpha\sin 2\phi].$$

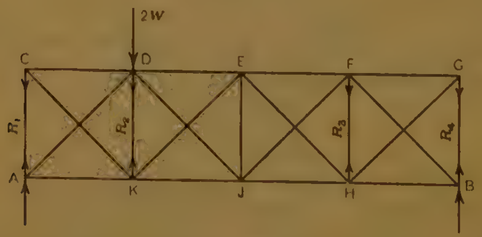
The values of the end-reactions are thus determined for the symmetrical and skew-symmetrical load-systems, and their superposition gives

$$M_A = M_b + M_c$$
$$M_B = M_b - M_c$$
$$T_A = T_b + T_c$$
$$T_B = T_b - T_c$$
$$F_A = W + F_c = 2W - F_D$$
$$F_B = W - F_c = F_D$$

Application to Braced Frames with Redundant Bars.

When a structure which has a number of redundant bars exhibits symmetry about a centre-line an application of the Principle of Superposition considerably reduces the work of analysis.

Figs. 10.



The example chosen to illustrate this is a simple one and is shown in *Fig. 10.* It consists of a four-panel truss with counterbracing in each panel, so that there are four redundant bars. A load of $2W$ acts at joint D as shown, and the bars CA, DK, FH and GB are

conveniently chosen as the redundant members, the tensions in them being R_1, R_2, R_3 and R_4 respectively.

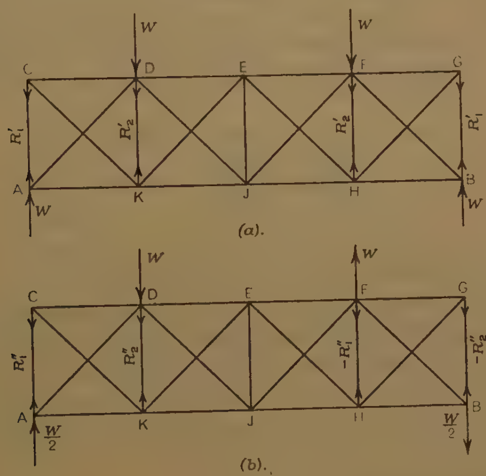
Using the second theorem of Castigliano, the conditions for the solution of the problem are

$$\frac{\partial U}{\partial R_1}=\frac{\partial U}{\partial R_2}=\frac{\partial U}{\partial R_3}=\frac{\partial U}{\partial R_4}=0.$$

The four resulting equations in the ordinary way have to be solved simultaneously.

Suppose, however, that the load-system is split up into the two systems shown in *Figs. 11.* The system shown in *Figs. 11* (*a*) is symmetrical about EJ, the centre-line of the truss, and so correspond-

Figs. 11.



ing bars on either side of this axis of symmetry must carry the same loads. It is only necessary, therefore, to assume two unknown tensions R'_1, and R'_2, as shown instead of four as in the case of unsymmetrical loading.

The second load-system shown in (*b*) is skew-symmetrical, a load W acting downwards at D and an equal load acting upwards at F. The reactions are as shown, and it is clear that corresponding bars on either side of the centre-line will carry equal but opposite loads. Thus, if CA has a tension R''_1, GB will have a tension of $-R''_1$; that is, a compression of $+R''_1$. Here again the number of statically indeterminate forces is two instead of four. If the load-systems shown at (*a*) and (*b*) are superposed the result is $2W$ acting at D as shown in *Fig. 10*, and the original problem is thus reduced to the

solution of two pairs of two simultaneous equations instead of the solution of four simultaneous equations.

The saving in work is considerable even in the present simple case, but in a problem with a larger number of redundant bars the method enables solutions to be obtained which would otherwise be impracticable. For example, twelve simultaneous equations would generally involve a prohibitive amount of work, and it is only in very exceptional circumstances that a solution would be attempted; if, however, by the use of the Principle of Superposition they can be reduced to two independent sets of six each the problem, although still lengthy, is quite practicable.

The example shown in *Fig. 10* has been worked out by the straightforward method and also by using superposed load-systems, and the working is given for the sake of comparison.

Numerical values have been reduced to the simplest terms and are not intended to represent an actual case, as it is desirable in illustrating a method to keep the arithmetical work as easy as possible.

In each case equations are formed of the type

$$\frac{\partial U}{\partial R} = \frac{P_0 L \partial P_0}{AE \partial R} = 0$$

where P_0 is the load in any member expressed in the form $P_0 = aW + \alpha R_1 + \beta R_2 + \ldots + \nu R_n$ where a, α, β . . . ν are numerical coefficients and $R_1 \ldots R_n$ are redundant forces, L denoting the length of the member and A its cross-sectional area. The material used is supposed to be the same throughout.

(1) First dealing with the problem straightforwardly, the necessary four simultaneous equations are formed, the work being set out in Table I.[1]

The equations obtained are

$$\left.\begin{array}{r}
9W + 40R_1 - 20R_2 - 2R_3 + 2R_4 = 0 \\
9W - 40R_1 + 44R_2 + 4R_3 - 4R_4 = 0 \\
-9W - 4R_1 + 4R_2 + 44R_3 - 40R_4 = 0 \\
9W + 2R_1 - 2R_2 - 20R_3 + 40R_4 = 0.
\end{array}\right\}$$

The solution of these gives

$$\begin{aligned}
R_1 &= -0{\cdot}59799W \\
R_2 &= -0{\cdot}77006W \\
R_3 &= +0{\cdot}01434W \\
R_4 &= -0{\cdot}22643W.
\end{aligned}$$

[1] For a detailed description of procedure reference may be made to A. J. S. Pippard and J. F. Baker, "Analysis of Engineering Structures," p. 95. London, 1936.

TABLE I.

Bar	A	L	P_0					$\frac{P_0L}{A}\ \frac{\partial P_0}{\partial R_1}$					$\frac{P_0L}{A}\ \frac{\partial P_0}{\partial R_2}$					$\frac{P_0L}{A}\ \frac{\partial P_0}{\partial R^8}$					$\frac{P_0L}{A}\ \frac{\partial P_0}{\partial R_4}$				
			W	R_1	R_2	R_3	R_4	W	R_1	R_2	R_3	R_4	W	R_1	R_2	R_3	R_4	W	R_1	R_2	R_3	R_4	W	R_1	R_2	R_3	R_4
AC	2	1		1					$\frac{1}{2}$																		
KD	2	1			1										$\frac{1}{2}$												
JE	2	1		-1	1	1	-1		$\frac{1}{2}$	$-\frac{1}{2}$	$-\frac{1}{2}$	$\frac{1}{2}$		$-\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$-\frac{1}{2}$		$-\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$-\frac{1}{2}$		$\frac{1}{2}$	$-\frac{1}{2}$	$-\frac{1}{2}$	$\frac{1}{2}$
HF	2	1				1															$\frac{1}{2}$						
BG	2	1					1																				$\frac{1}{2}$
CD	4	1		1					$\frac{1}{4}$																		
DE	4	1	-1	-1	1			$\frac{1}{4}$	$\frac{1}{4}$	$-\frac{1}{4}$			$-\frac{1}{4}$	$-\frac{1}{4}$	$\frac{1}{4}$												
EF	4	1	-1			1	-1											$-\frac{1}{4}$			$\frac{1}{4}$	$-\frac{1}{4}$	$\frac{1}{4}$			$-\frac{1}{4}$	$\frac{1}{4}$
FG	4	1					1																				$\frac{1}{4}$
AK	4	1	$\frac{3}{2}$	1				$\frac{3}{8}$	$\frac{1}{4}$																		
KJ	4	1	$\frac{3}{2}$	-1	1			$-\frac{3}{8}$	$\frac{1}{4}$	$-\frac{1}{4}$			$\frac{3}{8}$	$-\frac{1}{4}$	$\frac{1}{4}$												
JH	4	1	$\frac{1}{2}$			1	-1											$\frac{1}{8}$			$\frac{1}{4}$	$-\frac{1}{4}$	$-\frac{1}{8}$			$-\frac{1}{4}$	$\frac{1}{4}$
HB	4	1	$\frac{1}{2}$				1																$\frac{1}{8}$				$\frac{1}{4}$
CK	$\sqrt{2}$	$\sqrt{2}$		$-\sqrt{2}$					2																		
AD	$\sqrt{2}$	$\sqrt{2}$	$-\frac{3}{\sqrt{2}}$	$-\sqrt{2}$				3	2																		
DJ	$\sqrt{2}$	$\sqrt{2}$	$-\frac{1}{\sqrt{2}}$	$\sqrt{2}$	$-\sqrt{2}$			-1	2	-2			1	-2	2												
EK	$\sqrt{2}$	$\sqrt{2}$		$\sqrt{2}$	$-\sqrt{2}$				2	-2				-2	2												
EH	$\sqrt{2}$	$\sqrt{2}$				$-\sqrt{2}$	$\sqrt{2}$														2	-2				-2	2
FJ	$\sqrt{2}$	$\sqrt{2}$	$\frac{1}{\sqrt{2}}$			$-\sqrt{2}$	2											-1			2	-2	1			-2	2
FB	$\sqrt{2}$	$\sqrt{2}$	$-\frac{1}{\sqrt{2}}$				$-\sqrt{2}$																1				2
GH	$\sqrt{2}$	$\sqrt{2}$					$-\sqrt{2}$																				2

The negative signs denote that the forces are compressive.

(2) Using the method of load-superposition, and dealing first with the case shown in *Figs. 11* (*a*), the equations obtained are

$$\left.\begin{aligned} -9W - 42R'_1 + 22R'_2 &= 0 \\ 22R'_1 - 24R'_2 &= 0 \end{aligned}\right\}$$

which give

$$\left.\begin{aligned} R'_1 &= -0{\cdot}41221W \\ R'_2 &= -0{\cdot}37786W \end{aligned}\right\}$$

The detailed work is set out in Table II.

TABLE II.

Bar	A	L	P_0		
			W	R'_1	R'_2
CA	2	1		1	
DK	2	1			1
CD	4	1		1	
DE	4	1	−1	−1	1
AK	4	1	1	1	
KJ	4	1	1	−1	1
CK	$\sqrt{2}$	$\sqrt{2}$		$-\sqrt{2}$	
DA	$\sqrt{2}$	$\sqrt{2}$	$-\sqrt{2}$	$-\sqrt{2}$	
DJ	$\sqrt{2}$	$\sqrt{2}$		2	$-\sqrt{2}$
KE	$\sqrt{2}$	$\sqrt{2}$		2	$-\sqrt{2}$
$\frac{1}{2}$ (EJ)	$\frac{1}{2}$ (2)	1		−1	1

Bar	$\frac{P_0 L \partial P_0}{A \partial R'_1}$			$\frac{P_0 L \partial P_0}{A \partial R'_2}$		
	W	R'_1	R'_2	W	R'_1	R'
CA		$\frac{1}{2}$				
DK						$\frac{1}{2}$
CD		$\frac{1}{4}$				
DE	$\frac{1}{4}$	$\frac{1}{4}$	$-\frac{1}{4}$	$-\frac{1}{4}$	$-\frac{1}{4}$	$\frac{1}{4}$
AK	$\frac{1}{4}$	$\frac{1}{4}$				
KJ	$-\frac{1}{4}$	$\frac{1}{4}$	$-\frac{1}{4}$	$\frac{1}{4}$	$-\frac{1}{4}$	$\frac{1}{4}$
CK		2				
DA	2	2				
DJ		2	−2		−2	2
KE		2	−2		−2	2
$\frac{1}{2}$ (EJ)		1	−1		−1	1

The skew-symmetrical case shown in *Figs. 11* (*b*) yields the equations

$$\left.\begin{aligned} 19R''_1 - 9R''_2 &= 0 \\ -9W + 36R''_1 - 40R''_2 &= 0 \end{aligned}\right\}$$

the solution being

$$\left.\begin{aligned} R''_1 &= -0{\cdot}18578W \\ R''_2 &= -0{\cdot}39220W \end{aligned}\right\}$$

The numerical work is given in Table III.

The redundant forces under the original loading are then

$$\begin{aligned} R_1 &= R'_1 + R''_1 = -(0{\cdot}41221 + 0{\cdot}18578)W = -0{\cdot}59799W \\ R_2 &= R'_2 + R''_2 = -(0{\cdot}37786 + 0{\cdot}39220)W = -0{\cdot}77006W \\ R_3 &= R'_2 - R''_2 = -(0{\cdot}37786 - 0{\cdot}39220)W = +0{\cdot}01434W \\ R_4 &= R'_1 - R''_1 = -(0{\cdot}41221 - 0{\cdot}18578)W = -0{\cdot}22643W \end{aligned}$$

TABLE III.

Bar	A	L	P_0			$\frac{P_0 L \, \partial P_0}{A \, \partial R''_1}$			$\frac{P_0 L \, \partial P_0}{A \, \partial R''_2}$		
			W	R''_1	R''_2	W	R''_1	R''_2	W	R''_1	R''_2
CA	2	1		1			$\frac{1}{2}$				
DK	2	1			1						$\frac{1}{2}$
CD	4	1		1			$\frac{1}{4}$				
DE	4	1		-1	1		$\frac{1}{4}$	$-\frac{1}{4}$		$-\frac{1}{4}$	$\frac{1}{4}$
AK	4	1	$\frac{1}{2}$	1		$\frac{1}{8}$	$\frac{1}{4}$				
KJ	4	1	$\frac{1}{2}$	-1	1	$-\frac{1}{8}$	$\frac{1}{4}$	$-\frac{1}{4}$	$\frac{1}{8}$	$-\frac{1}{4}$	$\frac{1}{4}$
CK	$\sqrt{2}$	$\sqrt{2}$		$-\sqrt{2}$			2				
DA	$\sqrt{2}$	$\sqrt{2}$	$-\frac{1}{\sqrt{2}}$	$-\sqrt{2}$		1	2				
DJ	$\sqrt{2}$	$\sqrt{2}$	$-\frac{1}{\sqrt{2}}$	$\sqrt{2}$	$-\sqrt{2}$	-1	2	-2	1	-2	2
KE	$\sqrt{2}$	$\sqrt{2}$		$\sqrt{2}$	$-\sqrt{2}$		2	-2		-2	2

These results agree exactly with those obtained from the straightforward solution, but the work involved is very considerably less.

It should be noted that the calculations are only required for one-half of the truss under each of the two component systems. In the case of the skew-symmetrical loading the centre-post JE is unstressed and presents no difficulty. In the case of the symmetrical loading, however, it is necessary to take a post of one-half the actual area carrying one-half the load when dealing with the half truss. This is indicated in Table II by the description ½ (EJ) in the first column.

MULTIPLE LOADS ON A STRUCTURE.

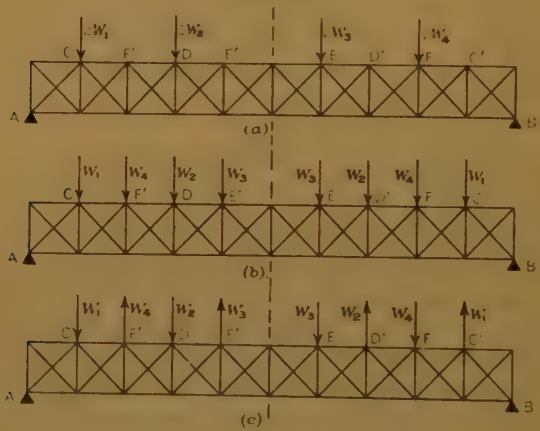
The cases illustrated have dealt either with continuous loading or with a single concentrated load. There is, however, no difficulty in applying the same method to any load-system.

In the case of a structure such as a beam, arch, or portal, it is only necessary to determine the reactions for a single point-load, since if the position of such a load is defined algebraically a general solution results which can be used to calculate the reactions for the component loads of the specified system. The sums of the reactions thus found are the required values when the whole load-system is applied simultaneously. This is, in fact, the simplest possible application of the Principle of Superposition.

In the case of a framed structure, however, this method is not generally practicable and the whole load-system must be treated together. The method is best illustrated by an example.

Suppose *Figs. 12* (*a*) represents a structure loaded as shown, the loads $2W_1$, $2W_2$, $2W_3$ and $2W_4$ being applied to the points C, D, E and F respectively. This system is to be replaced by symmetrical and skew-symmetrical systems which, when superposed, give the original system. Let C′, D′, E′ and F′ be the points corresponding to C, D, E and F on the other side of the centre-line of the truss.

Figs. 12.



Consider first the load $2W_1$. This is replaced by a pair of downward loads W_1 at C and C′ as shown in *Figs. 12* (*b*), and by downward and upward loads W_1 at C and C′ respectively as shown in *Figs. 12* (*c*). The other loads are dealt with similarly, and the two systems (*b*) and (*c*) result. The stress-analysis is then carried out as illustrated in the detailed case already given.

The Paper is accompanied by three sheets of diagrams, from which the Figures in the text have been prepared.

Paper No. 5151.

"The Water Supplies of Pahang."

By Edmund David Kibble, M. Inst. C.E.

(Ordered by the Council to be published in abstract form.) [1]

Pahang is a tropical State with approximately the same area as Wales, situated to the north of the equator and bounded on the east by the China Sea. The country is mainly mountainous and fairly well-drained by rivers and streams. Except in the hills, the temperature varies between a minimum of about 70° F. and a maximum of about 90° F. The average rainfall is about 107 inches and the average relative humidity is about 80 per cent.

The Paper describes the administration and control of water supplies by the Public Works and Medical Departments. The original policy was to impound only clear mountain streams for water supply, whereas the present procedure is to utilize the nearest available constant supply. There are thirteen town and village supplies in Pahang, for which some figures of cost and consumption are given. The supplies are divided into three main categories :—

(1) From clear hill-streams without treatment.
(2) From turbid hill-streams after treatment and filtration.
(3) From polluted rivers after treatment, filtration and sterilization.

Typical examples are described in each category, reports of chemical examinations of raw and treated waters are given, and some observations are made on the results of bacteriological examinations. Comparative abstracts of pumping and filtration costs, with notes on control of treatment and on chlorination, are added.

The Author submits the following conclusions :—

(1) Unless the cost of power is prohibitive, the best policy is to pump water from the nearest source where the quantity is sufficient at all seasons, and to obtain the required quality by treatment.

[1] The MS. and drawings can be seen in the Institution Library.—Sec. Inst. C.E.

(2) The ordinary civil engineer with general experience can obtain satisfactory results from rapid-filtration plants if he has acquired a working knowledge of the process of coagulation and of *p*H control. A specialized knowledge of the chemistry of water is not necessary, provided that the services of a competent chemical analyst are available for periodic examinations and for advice when required.

(3) Coagulation before filtration is an essential part of a rapid-filter plant.

(4) Sterilization by means of chlorine gas is unsatisfactory in the case of small water supplies where the apparatus is not in continuous operation.

(5) The covering of service reservoirs in the tropics is unnecessary, provided that they are situated in isolated positions with no direct risk of pollution from birds, animals, or wind-blown leaves.

The Paper is accompanied by six sheets of drawings descriptive of typical filtration plants and coagulating tanks, and by one map.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Committee on Soil Corrosion of Metals and Cement Products.

REFERENCE was made to the formation of this Committee in the December, 1936, Journal.[1] The Sub-Committee dealing with cement products has already drawn up a programme of research, and preliminary investigations in respect of suitable sites for field tests are now being carried out, as explained in the November, 1937, Journal.[2]

With regard to soil corrosion of metals, a considerable amount of research has been carried out abroad, particularly in America. The Committee therefore decided that the first step was to collect and make a study of existing information, and a Sub-Committee was formed for this purpose, consisting of:—

W. H. HATFIELD, D.Met., F.R.S. (*Chairman*).
J. C. HUDSON, D.Sc.
H. MOORE, C.B.E., D.Sc.

This Sub-Committee prepared a valuable digest of existing knowledge and a comprehensive bibliography, and also indicated where further research was necessary.

It was quite evident that variation in the nature and condition of the soil was the predominating factor in determining corrosion. The variation in composition of ferrous metals, for example, is of relatively minor importance. Before the question of experimental work could be usefully considered, it was felt that preliminary consideration should be given to questions of environment generally. A Sub-Committee was accordingly appointed for this purpose, consisting of:—

J. R. DAVIDSON, C.M.G., M.Sc. (*Chairman*).
ALFRED BRANDT, of the British Non-Ferrous Metals Research Association.
E. M. CROWTHER, D.Sc., F.I.C., of the Rothamsted Experimental Station.
H. G. DINES, of the Geological Survey.
J. C. HUDSON, D.Sc., of the Iron and Steel Institute Research Committee.
F. M. LEA, D.Sc., of the Building Research Station.
H. G. TAYLOR, M.Sc., of the Electrical Research Association.

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[1] Vol. 4 (1936–37), p. 310.
[2] Vol. 7 (1937–38), p. 153.

A questionnaire had previously been sent out seeking information in regard to soil corrosion due to the presence of sulphate salts. The information obtained is now being supplemented by means of a further questionnaire directed to those areas where deterioration of metals and concrete in the soil from causes other than sulphate salts might possibly be expected. The Institution will welcome any information on this point and a form of questionnaire will gladly be sent on request to anyone in a position to supply such data.

RESEARCH WORK IN ENGINEERING AT THE UNIVERSITY OF LEEDS.

The following notes refer to engineering research in the Departments of Civil, Mechanical and Electrical Engineering and to researches of engineering interest in the Departments of Fuel and Metallurgy and of Mining.

Civil Engineering.

During the last few years work has been conducted on the elastic and plastic strains in plain and reinforced-concrete beams and arches, the experiments on arches being pursued on three-hinged, two-hinged, and rigid arches. The distribution of horizontal, vertical and shearing stresses has been observed along vertical sections of beams, and the influence of concentrated or point loadings on these stress-distributions has been noted. The results obtained [1] show that the stresses in the tension and compression reinforcement under working loads are in fair agreement with those calculated, whilst the stress induced in the shear reinforcement, whether in the form of vertical stirrups, inclined stirrups or bent-up rods, is rarely as much as 75 per cent. of that calculated. The type of shear reinforcement is found to have a marked influence on the magnitude

[1] R. H. Evans, "Stresses in the Steel Reinforcement of Reinforced Concrete Structures." *The Structural Engineer*, vol. 13 (1935), p. 354.

—— "Strain and Stress Distribution in Concrete Beams." *The Engineer*, vol. 159 (1935), p. 94.

—— "Shear Stress Distribution in Reinforced Concrete Beams." *Engineering*, vol. 139 (1935), p. 158.

—— "Stresses in the Shear Reinforcement of Reinforced Concrete Beams." *Engineering*, vol. 142 (1936), p. 269.

—— "Experiments with Cold-Worked Steel as Reinforcement." *Concrete and Constructional Engineering*, vol. 32 (1937), p. 567.

of the deflexion. The neutral axis of zero strain in doubly-reinforced concrete beams [1] is found to fall appreciably during creep under sustained loads.

A large amount of research has been carried out on the elasticity and plasticity of natural and artificial stones when subjected to both very low and high stresses, and it has been possible to explain the peculiarities in the values of the moduli of elasticity under small stresses.[2] Further work [3] on lateral strain in concrete and other building materials has indicated that Poisson's ratio usually becomes abnormal at stresses exceeding about 50 per cent. of the crushing stress.

More recently, experiments have been conducted on the influence of moulding pressure on the compressive strength of concrete. The initial water/cement ratio is of little significance when concrete is subjected to high pressures in a mould; in fact, the very wet mixes gave the highest crushing strength.[4] It is also found that with pressures above from $1\frac{1}{2}$ to 2 tons per square inch the aggregate is damaged and the crushing strength falls, especially with the driest mixes. Experiments are now in progress on vibration-, as compared with pressure-, moulding.

Efforts to discover a fundamental invariable property, measurable at any time, in materials exhibiting both plastic and elastic strains have resulted in the isolation of a rate of strain [5] which has been termed "short-range strain."

Mechanical Engineering.

During the last few years the quantum specific heats derived from spectroscopic data for the gases present in the cylinder of an internal-combustion engine have been proved, by means of closed-vessel

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[1] R. H. Evans, "Experiments on Stress Distribution in Reinforced Concrete Beams." *The Structural Engineer*, vol. 14 (1936), p. 118.
—— "Strain and Stress Distribution in Doubly Reinforced Concrete Beams." *The Engineer*, vol. 164 (1937), p. 335.

[2] R. H. Evans and R. H. Wood, "Modulus of Elasticity of Materials for Small Stresses." *Phil. Mag.*, 7th series, vol. 21 (1936), p. 65.

[3] —— "Transverse Elasticity of Building Materials." *Engineering*, vol. 142 (1936), p. 409.
—— "Transverse Elasticity of Natural Stones." Proc. Leeds Phil. and Lit. Soc., vol. 3 (1937), p. 340.

[4] —— "The Influence of Moulding Pressure on the Compressive Strength of Concrete." *Concrete and Constructional Engineering*, vol. 32 (1937), p. 121.

[5] R. H. Evans, "The Elasticity and Plasticity of Rocks and Artificial Stone." Proc. Leeds Phil. and Lit. Soc., vol. 3 (1936), p. 145.

explosion experiments,[1] to be of a good degree of accuracy up to a temperature of 2,500° C. and are probably accurate up to at least 2,800° C. Those of H_2O above 2,000° C. were a little doubtful, but it now seems probable from experiments in progress that they may be used without serious error. The ideal efficiencies of gas and petrol engines are being calculated upon the basis of these specific heats, and a comparison is being made between them and actual measured efficiencies. It has been found necessary in calculating the ideal efficiencies to take into account the dissociations $H_2O \rightleftarrows OH + \frac{1}{2}H_2$ and $H_2 \rightleftarrows 2H$, as well as dissociations $CO_2 \rightleftarrows CO + \frac{1}{2}O_2$ and $H_2 + CO_2 \rightleftarrows CO + H_2O$.

Experiments have shown that that part of the heat-loss in high-speed petrol engines which affects the thermodynamic efficiency is far less than has hitherto been thought,[2] that as much as two-thirds of the heat which passes into the cooling water is given up during the violent rush of the exploded gases through the exhaust valve and passages,[3] and that the heat-loss during the explosion–expansion stroke in high-speed internal-combustion engines is unaffected by the turbulence resulting from rapid entry of the charge into the engine cylinder, this being of little significance compared with turbulence set up during combustion.

Experiments are in progress with the object of seeing whether the highest useful compression-ratio may be raised by design, and also whether it is possible to eliminate substantially the "delay period".

Flame-temperature measurements taken by means of fine platinum-wire thermometers have shown that a large amount of long-lived latent energy exists in flame gases, depending upon (i) the nature of the combustible gas, (ii) the nature of the diluent gases, and (iii) the instantaneous pressure in the flame-front.[4] Small quantities of certain gases are found to decrease the amount of latent

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[1] Prof. W. T. David and A. S. Leah, "The Specific Heats of Gases at High Temperatures." *Phil. Mag.*, 7th series, vol. xviii (1934), p. 307.

[2] —— "The Specific Heats of Gases at High Temperatures." *The Engineer*, vol. clxii (1936), p. 675.

[3] Prof. W. T. David, "The Distribution of Heat Losses in Petrol Engines." *The Engineer*, vol. clxiii (1937), p. 154.

[4] W. T. David, W. Davies, and J. Jordan, "Flame Temperatures." *Phil. Mag.*, 7th series, vol. xii (1931), p. 1043.

W. T. David and J. Jordan, "Flame Temperatures in Methane-air Mixtures." *Ibid.*, vol. xviii (1934), p. 228.

W. T. David and A. S. Leah, "Latent Energy in Explosions." *Ibid.*, vol. xxii (1936), p. 513.

Prof. W. T. David, "Latent Energy in Combustion." *Ibid.*, vol. xxiii (1937), p. 345.

energy very considerably. This appears to reside in some metastable state of the molecules formed during combustion in the flame-front, and experiments now in progress prove that the excess energy in the metastable molecules is unloaded when they come into contact with a surface. The latent energy in explosions and in internal-combustion engines has been found to be markedly less than that in flames,[1] due to the fact that as flame travels outwards from the spark the instantaneous pressure in the flame-front increases. Much of the effectiveness of certain industrial processes is believed to be due to the potency of this latent energy.

Electrical Engineering.

A research is being made into the oscillations in iron-cored circuits, with special reference to transformer neutral-point oscillations. In long transmission-schemes where line capacity plays an important part, it is not uncommon to find that high voltages, accompanied by oscillations, make their appearance when one earth connexion only is made to the star-point of a transformer bank. These oscillations have their origin in the variable inductance of the transformer cores, leading to resonance effects in conjunction with the line capacity. The object of this research is to study the conditions which lead to such phenomena. Oscillograms have been obtained for a variety of combinations of inductance and capacity. Peak voltages have also been measured and a theory has been evolved which may be applied to various circuit arrangements.

An experimental study is being made of stability in transmission schemes. A mathematical analysis of the dynamic conditions producing instability, when synchronous machines are joined by a network, is possible for two machines and a simple network, but the calculation, which must proceed by a step-by-step method, is laborious. Previous investigators of this problem have used purely mechanical apparatus, following the same differential equations as those governing the action of the electrical machines. The present research is concerned with the setting-up of a system containing small generators connected by a model network. The constants of the generators are measured and compared with those of actual machines.

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[1] W. T. David and A. S. Leah, "Latent Energy in Explosions." *Ibid.*, vol. xxii (1936), p. 513.
Prof. W. T. David, "Combustion Levels in Flame Gases." *Engineering*, vol. cxliv (1937), p. 531.

Electrical measurements with the oscillograph are made on the connecting network when sudden increments of load are applied, and under other transient conditions. The oscillations of the machines are measured simultaneously. The work is directed principally towards the perfecting of this technique.

A research is also in progress on the mechanism of thyratron-striking. Empirical development of gas-filled triodes has run ahead of any theory as to their operation. Experiments are in progress to test a tentative theory which was put forward by Wheatcroft [1] and has been given *prima facie* proof by earlier tests. The technique is directed to the measurement of pre-striking currents, and so to the analysis of the internal ionizing conditions.

Fuel and Metallurgy.

For many years research into problems connected with gas have been carried out under the Joint Research Committee of the Institution of Gas Engineers and Leeds University. The present work consists of an investigation of the use of oxygen and high pressure in producing complete gasification. By the use of oxygen in place of air dilution with nitrogen is avoided. The process has been divided into two stages: (1) experiments at atmospheric pressure in which coal or coke is burned in a mixture of steam and oxygen to produce a gas consisting of varying proportions of hydrogen, carbon monoxide and carbon dioxide, and (2) the synthesis of gaseous hydro-carbons by the heating of coke at high pressure. It is found that when heated under pressure in an atmosphere of hydrogen or of hydrogen and carbon monoxide the yield of gaseous hydrocarbons is considerably higher than can be produced in an atmosphere of nitrogen. In the practical application of this research the first and second stage would probably be combined into a single process, but it is advantageous that they should be studied separately.

A general study is being made of the corrosion of metals by flue-gases. In one research the composition of the gas is varied, the same metal being used throughout, and a complementary research is being carried out into the scaling properties of special steels and non-ferrous metals in furnace-atmospheres. The scaling is being studied at temperatures of 650° C., and of from 1,000° C. to 1,400° C. Research is also being carried out on the effect of the corrosive properties of the condensate produced from the products of combustion of coal gas. The worst conditions of corrosion obtain in a flue in which the upper end is cooled, so that the vapour from the flue-gas condenses,

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[1] Proc. Leeds Phil. and Lit. Soc., vol. 3 (1937), p. 396.

runs down, and then is re-evaporated, and so on continuously, so that the condensate contains an ever-increasing concentration of the products of combustion. Such attack is being studied with small-scale flues composed of various ferrous metals. Investigations on the control of viscosity of tars may have an important influence on the production of road-binders.

Mining.

The Shotfiring Sub-Committee of the Explosives in Mines Research Committee is carrying out at the University of Leeds a research programme dealing with the use of explosives underground.

The present series of experiments relate to the firing of shots simultaneously and to methods avoiding the disadvantages which have caused the legislature to prohibit simultaneous shotfiring except in certain specified instances. This work has entailed detailed examination of shotfiring practice and the carrying out of experiments in many coalfields. The question of the elimination of misfired shots has been studied, and the detailed examination of detonators in the field and in the laboratory has been a feature of the work.

Research has been carried out with regard to stemming materials used in shotfiring practice, and it has been found that certain sand-clay stemming mixtures are the best, both from the point of view of safety and of economy.

The study of shotfiring practice is being extended by the use of apparatus to enable shots to be fired safely from power mains, which practice is not generally permitted under the provision of the Coal Mines Act.

The foregoing researches are being carried out under the direction of Professor W. T. David, M.A., Sc.D., D.Sc., Professor of Civil and Mechanical Engineering; Mr. R. H. Evans, Ph.D., M.Sc., Lecturer in Civil Engineering; Professor E. L. E. Wheatcroft, M.A., Professor of Electrical Engineering and Dean of the Faculty of Technology; Professor J. W. Cobb, C.B.E., B.Sc., Professor of Fuel and Metallurgy; and Professor F. S. Atkinson, M.Eng., Professor of Mining.

REPORT OF THE ROAD RESEARCH BOARD FOR THE YEAR ENDED 31st MARCH, 1937.*

In the recently-published report of the Road Research Board, it is noted that close co-operation has been maintained with the Ministry of Transport, with the result that it has been possible to arrange facilities for the Laboratory results to be checked by practical tests as soon as possible. An investigation into the use of asphalt and bitumen in roads has been begun in co-operation with the Asphalt Roads Association, and will be on the same lines as the research already in hand on the use of tar. An investigation has also been commenced for the Rubber Growers' Association on the possibility of improving the quality of bitumen and tar for road surfaces by the addition of rubber. Certain researches previously carried out for the Road Research Board at the Building Research Station have now been started at the Road Research Laboratory; these include work on soil-mechanics, grading of aggregates, workability of concrete and the design of concrete road-slabs. The need to correlate investigations of road materials made with the special road-machine with the performance of the materials in practice, has led to the laying down of a large number of different types of road surfaces on the Colnbrook by-pass, where they can be conveniently kept under observation and compared with road-machine tests on similar surfaces at the Laboratory. Extended use is being made of the aggregate vibrator which was developed for proportioning the water/cement ratio of concrete in such a way as to obtain high uniform quality, and arrangements have been made with the County Engineer for practical trials on a large scale in Surrey. Road Research Bulletins Nos. 1 and 2 and Technical Papers Nos. 1 and 4 * have been published, dealing with the resistance to skidding, the control of concrete-quality, and the shape of aggregates.

Advantage was taken of an exceptional number of failures in embankments and cuttings to make examinations of the stability of slopes. It has to a large extent been possible to explain the failures by laboratory analyses of soil samples and the application of soil-mechanics.

It has been found that suitable gradings for concrete aggregates depend upon the cement-content used and the workability that is required, and a Table has been drawn up showing the values of these factors suitable for various purposes. For high cement-content

* Published by H.M. Stationery Office.

workability depends very little on the aggregate grading, but as the cement-content is decreased the grading becomes increasingly important.

In research on bituminous materials difficulties in obtaining reproducible results to a sufficient degree of accuracy have been overcome and a fundamental research is being carried out into the properties of the materials, particular attention being paid to the hardening which takes place with age.

A fundamental study has been made of the mode of failure of road surfaces, where the duration of the stresses involved is of the order of $\frac{1}{100}$ second. It is hoped to link up this work with a complementary study at the National Physical Laboratory of impact-forces imposed by vehicles. A parallel work for which arrangements are being made is a laboratory examination of the behaviour of the materials under transitory stresses.

In the study of skidding the sideways-force coefficient is not in general a constant, but with most surfaces tends to fall with speed and to reach a steady value at between 30 and 50 miles per hour. In others, however, a continuous reduction of coefficient has been found, and tests are therefore being arranged for even higher speeds. Surface-texture prints of the tire of a car on the road have been found capable of yielding information regarding the extent and nature of the contact between tire and road, and its variation with time, and they may possibly throw light on the sideways-force coefficient. A profilometer has been developed for the study of road-surface irregularities. The total upward vertical movement of a recording wheel, integrated and expressed as inches per mile, forms a useful index of the condition of the road. Thus the best roads show a movement of 75 to 85 inches per mile and a badly pot-holed road may give a figure of up to 800 inches per mile.

NOTES ON RESEARCH PUBLICATIONS.

Commencing with this number references to research publications will be tabulated according to the following scheme, sections 1 to 4 of which conform to the new series of "Engineering Abstracts" starting in January, 1938 :—

1.—Engineering Construction.

(1) Surveying.
(2) Engineering Physics.
(3) Operations and Methods.

(4) Structures :—
  (*a*) Earthworks, Foundations, etc.
  (*b*) Dams, Retaining Walls, etc.
  (*c*) Capacity Structures.
  (*d*) Bridges, Arches, Roofs, Hangars, etc.
  (*e*) Tunnels.
  (*f*) Buildings.
(5) Railways (excluding Rolling-Stock).
(6) Docks, Harbours, Canals (including the constructional aspect of Irrigation and Drainage), Rivers, and Coastal Works.
(7) Water Engineering :—
  (*a*) Supply ;
  (*b*) Power.

2.—Mechanical Engineering.

(1) Prime Movers.
(2) Transmission and Conversion of Power.
(3) Applications of Power.
(4) Measuring Instruments.
(5) Processes.
(6) Lubrication.
(7) Miscellaneous.

3.—Shipbuilding and Marine Engineering.

4.—Mining Engineering.

(1) Mine Plans (Surveys, Subsidence, Drainage).
(2) Methods of Working.
(3) Explosives.
(4) Mine Ventilation.
(5) Boring and Sinking (inclusive of Oil).
(6) Winding and Hauling Machinery.
(7) Preparation of Coal and Minerals for Market.
(8) Coking and By-Products.
(9) Research.
(10) Electricity as applied to Mining.

5.—Electrical Engineering.

6.—Other branches of Engineering :—

Aeronautical.
Automobile.
Illuminating, Heating, Ventilating, and Acoustic.
Chemical.
Miscellaneous.

7.—Engineering Materials.

8.—Miscellaneous.

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OBITUARY.

H.E. MARCHESE GUGLIELMO MARCONI, G.C.V.O., D.Sc., LL.D., Honorary Member, was born at Bologna on the 25th April, 1874, the second son of Giuseppe Marconi, an Italian country gentleman, his mother being a daughter of Mr. Andrew Jameson, of Daphne Castle, Co. Wexford; he died in Rome on the 20th July, 1937. He was educated at Leghorn and at the University of Bologna. In 1895 he conceived the idea that a system of telegraphy through space could be provided by means of electric waves, the existence of which had been foreseen mathematically by Clerk Maxwell in 1864, and later investigated experimentally by Hertz, Oliver Lodge, Righi, and others. Although not the discoverer of electric waves, it is due to the genius and perseverance of Marconi more than of any other worker that this method is now one of the most important means by which communications are sent by telegraph and telephone to all parts of the earth.

After conducting a number of experiments at his father's country house near Bologna, he came to England in 1896, where he demonstrated his invention before officers of the Post Office and Foreign Government Departments, and was fortunate to enlist the interest of Sir William Preece, the Engineer-in-Chief of the Post Office, who gave him substantial assistance. Successful tests between St. Martin's-le-Grand and the Thames Embankment were followed by transmission for a distance of 4 miles across Salisbury Plain, and in 1897 he effected communication across the Bristol Channel, nearly 9 miles. In demonstrations carried out for the Italian Government in the same year, at Spezia, signals were sent for a distance of 12 miles. Wireless telegraphy was first used for commercial purposes in 1898, when the Kingstown Regatta races were reported by Marconi by means of wireless apparatus installed on a tug which followed the yachts on the Irish Sea. In the same year permanent stations were erected near the Needles, in the Isle of Wight, and at Bournemouth, and it was this method that was also successfully used for sending messages between South Foreland lighthouse and East Goodwin lightship, this installation being the means, during the 2 years it was in operation, of saving several vessels and a number of lives. In March, 1898, Marconi established communication by wireless across the English Channel, between England and France,

whilst in 1901 he succeeded in transmitting and receiving signals from the long-distance wireless telegraph station at Poldhu, in Cornwall, to St. John's, Newfoundland. In 1905 he took out his patents for the horizontal directional aerial, which marked an advance in the design of long-distance stations. In 1907 a limited service of press messages was inaugurated between Clifden in Ireland and Glace Bay in Canada, ordinary traffic being accepted a little later.

Although in the earliest experiments comparatively short waves had been employed, it had come to be generally accepted that for long-distance transmission long waves were essential, and accordingly wave-lengths measured in thousands or tens of thousands of metres were adopted for stations with any considerable range of working. About 1916 Marconi turned his attention in the opposite direction, to wave-lengths measured in tens of metres, and the practical outcome of this work was that the British Government accepted his company's proposals to employ the short-wave beam system for Imperial communications and to build stations capable of working with Canada, South Africa, India, and Australia. Later, about 1931, Marconi started a systematic investigation of the properties and characteristics of still shorter waves, of less than one metre in length, and the results so far obtained from his experiments appear to justify his confidence in the practicability of their use.

The value of Marconi's work has been recognized by Governments, universities, and learned societies all over the world. Amongst the many scientific awards granted to him may be mentioned the Nobel Prize for Physics in 1909, the Albert Medal of the Royal Society of Arts, the Franklin Medal, the John Fritz Medal, and the John Scott Medal, awarded him in the United States for "the invention of wireless telegraphy." His academic distinctions included honorary degrees from Oxford, Cambridge, Aberdeen, Glasgow, and Liverpool, and the Rectorship of St. Andrews. He delivered the James Forrest Lecture at The Institution of Civil Engineers in 1926, having been made an Honorary Member of The Institution in 1925. He was also awarded the Kelvin Medal in 1932. He was an honorary Grand Cross of the Royal Victorian Order, whilst in Italy he was a Grand Officer of the Order of St. Maurice and St. Lazarus and a Chevalier of the Civil Order of Savoy, and also held the Grand Cross of the Order of the Crown of Italy. He was also a Senator of the Kingdom of Italy, a Member of the Grand Council of the Fascisti, President of the Royal Academy of Italy and President of the Research Council, and received the hereditary title of Marchese in 1929. Brazil, China, Japan, Russia and Spain were other countries which conferred high distinctions on him.

He married in 1905 the Hon. Beatrice O'Brien, daughter of the

late Lord Inchiquin, by whom he had one son and two daughters. In 1927 he married, as his second wife, the Countess Bezzi-Scali of Rome, by whom he had one daughter.

The Right Hon. Lord RUTHERFORD, O.M., D.Sc., F.R.S., Honorary Member, was the son of Mr. James Rutherford, and was born at Brightwater, near Nelson, New Zealand, on the 30th August, 1871; he died in Cambridge on the 19th October, 1937. His early education was received at a school near Brightwater, at which he won a scholarship taking him to the secondary school at Nelson. Another scholarship enabled him to enter Canterbury College, Christchurch, where in 1893 he was awarded the M.A. degree with first-class honours in mathematics and physics; in 1894 he was awarded the degree of B.Sc. and an 1851 Exhibition scholarship, which enabled him to enter Trinity College, Cambridge, to work under Sir Joseph Thomson. He had already constructed a wave-detector based on the effect of oscillating currents upon a highly-magnetized needle, and he continued this work at Cambridge. Sir Joseph Thomson was at that time engaged at the Cavendish laboratory on research on the ionization of gases, and Rutherford gave him considerable assistance; the work led Rutherford to a series of researches on radio-activity which had a revolutionary effect on scientific thought. He had been appointed to the Macdonald Chair of Physics at McGill University, Montreal, in 1898, and the greater part of these earlier researches were carried out there. In 1901 he was awarded the degree of D.Sc. by the University of New Zealand, and subsequently received honorary degrees from many Universities throughout the world. In 1907 he accepted the Langworthy Chair of Physics at the University of Manchester, and there he carried on his researches into radio-activity and established the existence of the atomic nucleus. Finally, in 1919 he returned to Cambridge to fill the Cavendish Professorship, previously held by Sir Joseph Thomson, and thenceforward his research was mainly devoted to the study of the atomic nucleus.

His contribution to physical science included two hypotheses of great importance, together with the experimental work by which he

established their truth. The first assumption was that a natural transmutation of the elements was constantly taking place, and he showed that the observed effects of radio-activity were a consequence of the disintegration of the atom, which postulated a transmutation of the elements. The second hypothesis was that atoms themselves resembled solar systems, the central sun being represented by a positive nucleus in the atom, and the planets being represented by negative electrons revolving around the nucleus in orbits. This hypothesis was also proved by experimental work. He found that atoms could in some cases be split by bombarding them with the helium atoms resulting from the radio-active process, and in recent years many elements have been artificially disintegrated by the agency of various forms of radiation.

He was knighted in 1914, and was created Baron Rutherford of Nelson in 1931. He received the Nobel prize for chemistry in 1908, and the Royal Society awarded him the Rumford medal in 1905 and the Copley medal in 1922. He received the Franklin medal of the Franklin Institute in 1924, and the Faraday medal of the Institution of Electrical Engineers in 1930. His services to science and the State were recognized in 1925 when the Order of Merit was conferred on him. He was president of the British Association in 1923, and of the Royal Society from 1925 to 1930. In 1927 he succeeded Sir Joseph Thomson as Professor of Natural Philosophy at the Royal Institution. He was elected an Honorary Member of The Institution on the 24th April, 1928.

He married in 1900 Mary, only daughter of Mr. Arthur Newton, of Christchurch, New Zealand, who survives him, and by whom he had one daughter, now deceased.

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The Institution of Civil Engineers.

REPORT

OF

THE RESEARCH COMMITTEE

FOR THE YEARS 1935–36 AND 1936–37.

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THE RESEARCH COMMITTEE

OF

THE INSTITUTION OF CIVIL ENGINEERS.

1935–36.

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1936–37.

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Professor Gilbert Cook, D.Sc.
H. L. Guy, F.R.S. (*Since resigned*).
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R. G. Hetherington, C.B., O.B.E., M.A.
Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S.

* Elected Honorary Member of The Institution in March 1937.

Professor A. H. Jameson, M.Sc.
David Lyell, C.M.G., C.B.E., D.S.O.
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Sir Leopold Savile, K.C.B.
R. H. H. Stanger.
R. E. Stradling, C.B., M.C., Ph.D., D.Sc.

Secretary :—
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PART I: REPORT OF THE RESEARCH COMMITTEE.

In February, 1935, the Council of the Institution of Civil Engineers decided on a more active policy in regard to Engineering Research and to this end they constituted the Research Committee as an Institution committee with wide terms of reference.

In the past, the policy of The Institution in carrying out the principal object of the Charter, namely, the general advancement of Mechanical Science, has been to rely mainly on the recording, discussion, and publication of the results of the work of individual members. From time to time specific problems have been investigated by committees appointed specially for each case and by the allotment of Institution funds. Notable cases of this kind were the Heat Engine Trials Committee appointed in 1903, the Committee on Reinforced Concrete in 1908, and in more recent years the Sea-Action Committee, the Committee on Floods, and the Committee on Compressed Air Working, whose work produced results of great importance. The growth in scientific knowledge and in the size and complexity of the problems which confront the civil engineer, no less than the great changes which have taken place in the character of the materials with which he has to work, has brought about a realization of the necessity for greater understanding of the scientific basis of engineering practice.

In their time the great fathers of the engineering profession were in a sense research workers. Their knowledge and experience were built upon a foundation of practical experiment, informed by an accumulation of traditional skill and craftsmanship. Their successors throughout the nineteenth century began to rely more and more on scientific knowledge, finding themselves in fact to be

practising the application of pure science to the requirement of mankind. The recent great development of scientific method, the instruments of high precision that are now available, and the resources of the research worker now provide the engineer with entirely new powers with which to analyse and solve his problems and to guide his design. It is the utilization of these new powers which now generally goes under the name of engineering research.

Much advance has been made in engineering knowledge by individual engineers and by scientific workers in the Universities and Government and other laboratories; and work of a more corporate nature has been carried out by several of the specialized engineering Institutions, the results of which have had a profound effect on engineering practice in many branches. It appeared to the Council that a large field of engineering was as yet uncovered by these activities and that the resources of The Institution should, in pursuance of the objects of the Charter, be made available for work in such fields.

Terms of Reference.

The Council accordingly, in establishing the Research Committee, gave it wide terms of reference not wishing to exclude, by a more limited scope, work in any branch of engineering which might benefit from such assistance as Institution funds and membership could give. The Committee was entrusted with the duties of making recommendations of subjects for research and investigation, the formation of suitable committees to carry out such work, the direction and co-ordination of the work of these committees, and recommending the form in which results of research are to be published. The Committee was also given the duty of maintaining contact with engineering research work of other institutions, societies and bodies and of dealing with all matters arising out of British Standard Specifications.

Composition of Committee.

The Committee consists of twenty-two members, of whom twelve are members of Council and the remainder corporate members of The Institution, of whom no less than five must be persons engaged in engineering research. The Committee is appointed by the Council yearly at the first meeting of the Council in each new session. Membership of committees and sub-committees appointed to carry out and direct research under the general supervision of the Research Committee is not confined to membership of The Institution.[1]

[1] For Membership of Committees, *see* Appendix I (p. 30).

General Policy.

The Committee, while endeavouring to keep in touch with engineering research in all branches, has mainly limited its activities to problems in branches of engineering not ordinarily covered by the programmes of specialized engineering Institutions. But wherever such a problem appeared to be of interest to other Institutions, co-operation has been invited and this has resulted in active participation in the work by other Institutions, by nominating members to the directing sub-committees and in some cases by financial assistance. In certain cases joint committees with other Institutions have been formed as the result of such co-operation.

The Committee has adopted the general policy of making use of existing research organizations such as those of the Department of Scientific and Industrial Research, the Universities and Technical Colleges, and has made no attempt to set up any research organizations of its own. The Council has, however, placed at the disposal of the Committee a technically qualified member of the Institution staff, appointed specially for the purpose, who carries out the secretarial work of the Committee and of the various sub-committees.

Co-operation with Other Bodies.

The Committee has explored the possibility of closer co-operation with the other engineering bodies directed towards a pooling of resources in the common interest of the advancement of engineering knowledge, but so far it has not been found possible for a variety of reasons to achieve more than co-operation in dealing with *ad hoc* problems. It will be seen, however, from the more detailed reports which follow that this has resulted in active co-operation in investigating individual problems with the Institution of Electrical Engineers, the Institution of Structural Engineers, the Institution of Water Engineers, the Federation of Civil Engineering Contractors, the British Waterworks Association, the Water Companies' Association, the Iron and Steel Institute, the British Non-Ferrous Metals Research Association, and the British Committee on Large Dams of the World Power Conference.

The Committee wishes to acknowledge its indebtedness to the large number of Institutions and bodies that have given or promised assistance in respect of the various researches, either financially or by the provision of facilities and materials. A detailed list of contributors is included in Appendix II.

FINANCE.

On the recommendation of the Committee, the Council decided in July, 1935, to set aside a specific sum annually for expenditure on research. The amount thus made available was £1,000 in 1935–36 and £2,000 per annum thereafter, excluding the cost of staff and administration. The Council reserved the power to reconsider this level of expenditure at the end of 5 years. Whilst in October, 1937, the expenditure from the Research Reserve Fund authorized to be spent up to March, 1938, was £4,276 out of £6,200 provided by the Institution, this represents but a small proportion of the total expenditure on the various research problems during the period the Committee has been in existence, generous contributions being given by other bodies and particularly by the Department of Scientific and Industrial Research.

CLASSIFICATION OF PROBLEMS.

The researches may be classified under the general headings: Materials, Soil Mechanics, Hydraulics, Structures, and Specialized Engineering Practice and are as follows :—

Materials.

Soil Corrosion of Metals and Cement Products.
Vibrated Concrete.

Soil Mechanics.

Earth-Pressures.
Pile Driving.

Hydraulics.

Velocity Formulas.
Wave-Pressures.

Structures.

Dams : Special Cements for Large Dams.
Fish-Passes.

Reinforced Concrete : Reinforced-Concrete Structures for the Storage of Liquids.

Steelwork : Simply Supported Steel Bridges, including research on Wind-Pressures.
Steel Structures.
Repeated Stresses in Structural Elements.

Specialized Engineering Practice.

Earthing to Metal Water-Pipes and Mains.
Breathing Apparatus for Use in Sewers.

Detailed accounts of the progress of these researches up to October 31st, 1937, are given in Part II of this Report.

None of these investigations has as yet been completed. In connexion with the research on Vibrated Concrete, an Interim Report embodying the first year's work has been published.[1] The drawing up of a code of practice for Reinforced-Concrete Structures for the Storage of Liquids is well advanced, and the recommendations for regulations in respect of Earthing to Metal Water-Pipes and Mains have been completed and await approval by the bodies concerned. The draft recommendations for Breathing Apparatus for Use in Sewers have been completed with the exception of the specification of apparatus, which awaits the completion of research being carried out at Birmingham University.

British Standard Specifications.

The Committee, in the course of its work of studying Draft British Standard Specifications, has taken up the general question of the relations of The Institution to the British Standards Institution. It appeared to be of importance that, if The Institution were to be of fullest assistance to the British Standards Institution, there should be opportunities of seeing and commenting on draft specifications at an early stage. The Institution is represented on the Council of the British Standards Institution and is represented on the Engineering Divisional Council. As the result of negotiations with the British Standards Institution it has now been arranged for The Institution to be represented on the Building Divisional Council, the Mechanical Industry Committee, and the Iron and Steel Industry Committee. Whilst it is probably difficult for The Institution, as such, to be consulted or to express any corporate opinion on Draft Specifications, there is no doubt that there is considerable value to the British Standards Institution in having comments passed on to its various committees with the authority of a scientific committee such as the Research Committee. This aspect of the problem of dealing with Draft Specifications has been fully recognized by the British Standards Institution, and it is satisfactory to record that there has thus been found a means of valuable co-operation between the two bodies.

31*st January*, 1938.

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[1] Journal Inst. C.E., vol. 5 (1936–37), p. 435. (March 1937.)

PART II: REPORTS ON THE RESEARCHES.

Soil Corrosion of Metals and Cement Products.

The corrosion of metals in soils is a familiar subject, but, until recent years, probably most engineers had regarded concrete buried in the ground as free from all danger of corrosion, except, perhaps, under rare and artificial conditions. The increasing occurrence of reports of severe corrosion of concrete in clay soils containing sulphate salts has, however, shown the need for far more information as to the conditions which are potentially dangerous and the cheapest effective protective measures. Corrosion of metal pipes in clay soils of this type has also often been reported, and it was with these two facts in mind that a Sub-Committee on the Effect of Sulphate Salts on Concrete in general and on Metal Pipes was appointed in 1935. A questionnaire was circulated to engineers throughout the country inquiring into the prevalence of corrosion and the type of soil in which it was found. The Sub-Committee also proceeded to draw up a programme of research on the deterioration of concrete. At this stage it became evident that the research ought to be organized on a wider basis so as to include all corrosive soil conditions and so as to permit the reorganization of the Sub-Committee to include representatives of the Corrosion Committee of the Iron and Steel Institute and the British Non-Ferrous Metals Research Association. The latter had already carried out some preliminary research into soil corrosion and the former had similar research under consideration. In addition, the Electrical Research Association had in progress a research, in connexion with the question of earthing, on corrosion of metals in various soils with and without the addition of common salt, and it was considered that they should be represented on the committee.

A new Committee was therefore formed to include members from the foregoing bodies and, in addition, from the National Physical Laboratory, the Building Research Station and the Rothamsted Experimental Station. The terms of reference are as follows :—

> "To consider how knowledge of Soil Corrosion of Metals and Concrete can best be advanced, to propose research as may be necessary, and to allocate this to such organizations as are willing to undertake the work. To provide the necessary co-ordination in such research and to prevent any duplication or overlapping therein."

At the same time an additional Sub-Committee was formed to deal specifically with the corrosion of Cement Products. This Sub-Committee has revised the programme of experimental work drawn up by the earlier committee, and it has received offers of considerable financial support from manufacturers and organizations interested in cement and concrete. The research envisaged will extend over 10 years. Before making a final proposal for this research it was decided to make, in conjunction with the Building Research Station, a preliminary investigation of sites suitable for the burial and exposure of test specimens, in order that some closer estimate of the total probable cost could be reached.

A step preliminary to the examination of the corrosion of metals, both ferrous and non-ferrous, has been the formation of a small Sub-Committee to collect existing information and make recommendations as to the investigations which are required. This Sub-Committee reported in October, 1937, having made an exhaustive and valuable summary of existing research into corrosion.

From a study of work already recorded it became evident that the corrosion of metals was far more dependent upon the nature of the soil, its composition and physical condition in respect of perviousness, moisture-content, etc., than upon any variations in type of ferrous metal, or in the state of a non-ferrous metal. It was considered to be a necessary preliminary to the consideration of tests on various metals to concentrate attention on conditions of environment, and a Sub-Committee has accordingly been formed for this purpose.

Vibrated Concrete.

In recent years, considerable attention has been paid to the use of vibratory methods of compacting concrete although relatively little scientific study of the method has so far been made. It is every-day practice when packing granular materials to shake the container, so causing the grains to pack together into a denser mass, and it is reasonable to suppose that if freshly-mixed concrete is vibrated suitably, the particles of cement and sand might similarly interlock more closely than would be possible with ordinary hand-compacting. On the basis that a dense concrete is synonymous with a strong concrete, vibration-compacting may therefore be of considerable value.

It will be realized at once that if a wet mix is used, the maximum consolidation can often be obtained by very little compacting, and the use of vibration is not likely in practice to increase the quality of the concrete in such cases. In fact it is possible that the increased

motion given to the particles of the mix by vibration-compacting may be a serious disadvantage, as it may lead to segregation of the materials. With a very dry mix on the other hand, satisfactory consolidation is impossible by ordinary hand-ramming, but may be obtainable with suitable vibration. In this case the use of vibration is very effective in producing a dense, strong concrete.

Regarding the type of vibration to be applied in order to obtain the most successful and economical results there are three main factors to be considered :—

(1) The frequency of the vibration.
(2) The acceleration imparted to the concrete.
(3) The power required to maintain the vibration.

These three factors must be considered in relation to the time required to consolidate the concrete and the subsequent strength and surface appearance of the concrete. At the same time, information is required as to the best type of vibrator to use for particular jobs. The main types now available on the market are :—

(1) External vibrators applied or clamped to the shuttering.
(2) Internal vibrators placed in the concrete itself. These are usually so dimensioned that they rise automatically when the concrete is set in vibration.
(3) Surface vibrators which are moved about on the surface of the freshly poured concrete. They consist of slabs upon which one or more vibrating units are fixed.
(4) Table vibrators used for pre-cast products.

In May, 1935, a Sub-Committee of The Institution was set up to consider " the best means of investigating the effect of shaking, percussion, and vibrating of concrete during deposition, with particular reference to the effects on setting and ultimate qualities of the concrete." It was learned that the Institution of Structural Engineers had also formed a committee to consider the same question, and it was decided, therefore, to form a joint committee of the two bodies.

At this time the problem had also been considered at the Building Research Station, and in fact a table vibrator had been designed for vibrating up to 2 cwt. of concrete under certain controlled conditions of frequency and acceleration. It was therefore convenient that the Sub-Committee should co-operate with the Building Research Station in an experimental investigation of the effects of vibration-compacting. Accordingly this work was put in hand and started in April, 1936.

The investigation falls naturally into two parts :—first, a study of

the effects of varying the frequency, acceleration, and duration of vibration on the strength and other properties of concrete, and second, a study of the use of vibrators of all kinds in practice. The first part has now been completed; an interim report on the work appeared in the March 1937, Journal[1] of The Institution and a second report appears in the April 1938, Journal.[2]

The early work showed that the acceleration imparted to the mass of concrete is of much greater importance than the frequency of vibration as regards the ultimate strength of the concrete. So long as the vibration is sufficient to consolidate the concrete, there is, however, a tendency for a better surface appearance to be obtained with high-frequency vibration (up to 8,000 vibrations per minute). Most of the tests have been made with a frequency of 3,000 vibrations per minute, and all with the table vibrator mentioned above, using normal Portland cement.

The acceleration and duration of vibration necessary for proper consolidation vary considerably for different concrete mixes, but in most cases so far tested, there has been little gain in strength when the acceleration has been increased from $5g$ to $10g$ or the duration from 3 minutes to 12 minutes. Some interesting results were obtained in this connexion with a 1-to-6 mix with a water/cement ratio of 0·60 (by weight). When this mix was vibrated continuously for several hours the strength subsequently obtained was increased considerably (70 per cent.) beyond that obtained with 2 minutes' vibration, and it was thought that the vibration might be affecting the setting process of the concrete. Subsequent tests, however, have shown that the results were due to a decrease in the water/cement ratio of the mass of the concrete, as water was brought to the surface by the vibration and allowed to run off.

In all the work so far carried out, it appears that the properties of vibrated concrete are merely the properties of a concrete in which satisfactory consolidation has been obtained with mixes which are drier than those associated with hand-compacting. The method of vibration-compacting is one which extends the range of water/cement ratios towards lower limits than previously practicable. The strength and density can thus be improved, shrinkage and creep are reduced, and the bond with reinforcing steel is increased.

It should be realized that the greater compactness of vibrated concrete with reduced water/cement ratios must lead to an increase in the quantities of dry materials required for a given volume of concrete.

It is important that, when vibration-compacting is used, the

[1] Journal Inst. C.E., vol. 5 (1936–37), p. 435. (March 1937.)

[2] *Ibid.*, vol. 8 (1937–38), p. 553. (April 1938.)

mix should be designed accordingly. The use of a wet mix is undesirable, leading to segregation, and it may also result in very poor bond with reinforcing steel, particularly with horizontal reinforcement, a layer of water and slurry being collected at the bar-surface. With regard to aggregate-grading, it appears that from strength considerations the grading is unimportant if the concrete can be well consolidated with the required water/cement ratio. Some gradings will, however, necessitate the use of more water or, alternatively, an increase in the power in order to compact properly, and some attention to grading is desirable. Generally speaking, with vibration-compacting the grading can with advantage be rather coarser than would be used with hand-compacting.

In order that the second part of the investigation (a study of vibrators in practice) may be carried out in quantitative terms, it is necessary to devise an instrument by means of which the characteristics of the vibration of any part of the concrete may be measured. Such an instrument has already been designed, and a few trials have shown it to be fairly satisfactory, although it is rather large for use in the field; other smaller instruments are, therefore, being designed.

Earth-Pressures.

In 1925, at the Southampton meeting of the British Association, a Committee was appointed to study the question of Earth-Pressures. From 1927 to 1934, an experimental investigation was conducted by Professor C. F. Jenkin, F.R.S., initially at Oxford University and later at the Building Research Station. This work was carried out under the ægis of the Department of Scientific and Industrial Research and in association with the British Association Committee on Earth Pressures. It culminated in two reports in 1934 on "The Mechanics of Granular Materials" and "The Experimental Investigation of the Mechanics of Clay." The work is also recorded in the Proceedings of the Royal Society,[1] the Minutes of Proceedings of The Institution,[2] and in *Engineering*.[3] At this stage Professor Jenkin retired. His work was essentially a fundamental investigation into the mechanics of granular material, and his later work was directed towards the investigation of the properties of kaolin.

In January, 1936, by arrangement with the British Association

[1] C. F. Jenkin, "The Pressure exerted by Granular Materials." Proc. Roy. Soc. A., vol. 131 (1931), p. 53.

[2] —— "The Pressure on Retaining Walls." Minutes of Proceedings Inst. C.E., vol. 234 (1931–32, part II), p. 103.

[3] —— "Predicting the Internal Motion of Sand." *Engineering*, vol. 133 (1932), p. 561.

and the Building Research Station, the British Association Committee on Earth Pressures was replaced by a Sub-Committee of the Institution Research Committee. The research which is now being carried out at the Building Research Station follows more closely on the lines along which the modern science of Soil Mechanics is being developed. It is concerned with the investigation of the mechanical properties of soils as they exist in nature, and with the relation between the behaviour of structures and the properties of the soil on which they are built.

The problems which are being studied can be divided into two main groups, (1) those concerned with the settlement of structures, (2) those concerned with the stability of earth slopes.

In the first group the most important problem is that of the settlement of buildings. The study of this problem involves (*a*) the recording of settlement observations, (*b*) the exploration of soil-strata, (*c*) the laboratory examination of subsoil samples, (*d*) the estimation of the pressures in the subsoil due to the building load, and (*e*) the computation of theoretical settlements on the basis of current theories. Observations of the movements of a building are taken in sufficient detail to enable settlement contours to be drawn at various stages during construction and at intervals extending over a period of time after the building has been completed. The exploration of the soil-strata at a site is carried out by means of well-boring apparatus, the number of bore-holes and the depth to which they are sunk depending on the size and loading of the building. In addition "undisturbed" samples are obtained from various depths by means of a special tool used in conjunction with the well-boring apparatus, and these are used for laboratory examination. The laboratory apparatus used is similar to the Terzaghi œdometer, and from the measurements taken the consolidation characteristics of the materials are determined. The distribution of vertical pressures in the subsoil due to the building load is calculated on the assumption of the applicability of the Boussinesq theory, and the theoretical settlement of the building due to vertical consolidation of the soil is then computed from the consolidation characteristics of the various soils within the sphere of influence of the building load.

The comparison between theoretical settlements and observed values is, of course, an important part of the research. It has already been possible to compare observed settlements with the theoretical value of the settlement of a building on London clay, and promising results have been obtained. The present programme includes observations of a number of buildings in various parts of the country. The buildings are of similar design and it is hoped that

the results of these investigations will make it possible to check the applicability of present methods of settlement analysis to a range of soil types.

The second group of problems is concerned with the stability of embankments, cuttings, retaining walls, dams, etc. The most important investigations which have so far been carried out have been concerned with the examination of shear slides in cuttings and embankments. The examination of shear-slide failures in earth slopes involves (*a*) the recording of data concerning the shape of the sliding surface, (*b*) the exploration of the soil-strata both in the earth bank and in the foundations below the bank, (*c*) the laboratory examination of the shear characteristics of the various soil strata, and (*d*) the stability analysis of the bank.

The recording of data which will assist in defining the shape of the slip-surface is of importance, since most of the methods of stability analysis assume that this surface is of cylindrical form. Whilst there is evidence to support this assumption when a slide occurs in fairly homogeneous soils, an appreciable departure from the cylindrical form may occur when the soil-strata are of widely different types. In exploring the soil-strata it is again important to obtain "undisturbed" soil samples for laboratory examination.

The investigation of the shear characteristics of soils still calls for research, and the application of the results of laboratory tests to practical problems is limited in some cases by the element of uncertainty in interpreting the test results. Three types of test are used: (1) shear box, (2) ring torsion, (3) unrestrained cylinder compression. The apparatus for the first test is similar to the Krey-Casagrande apparatus and the shear strength of the soil is determined in contact with water under various loads. The other two tests have been described in the Paper on "The Shearing Resistance of Soils," mentioned later.[1]

A number of methods for analysing the stability of earth banks have been published, but probably the most useful of these is the method originated by Petterson and developed by Fellenius, Krey, Terzaghi, and others. This method assumes that failure takes place on a cylindrical surface and considers the stability of this cylindrical segment against rotation about the axis of the cylinder.

By the courtesy of railway companies and other authorities several cases of slips in cuttings and embankments have been investigated, and the stability analyses which have been carried out using the results obtained by laboratory tests have given confidence in the applicability of the methods to problems of this nature.

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[1] Footnote (1), p. 16.

A list of practical problems which it is considered should be embraced by the scope of the research is being drawn up, and a summary of present knowledge of soil mechanics is under preparation at the Building Research Station. The following Papers dealing with the work since The Institution has taken an active part have appeared in the Journal.[1]

Pile Driving.

In 1932 research on the stresses set up during the driving of reinforced-concrete piles was commenced at the Building Research Station. The work was carried on as a co-operative investigation in collaboration with the Federation of Civil Engineering Contractors, and was initiated as the result of difficulties experienced by contractors in driving piles under certain conditions of hard driving, where it was found impossible to comply with some specifications and at the same time to avoid serious damage to the piles. Excavation sometimes revealed unsuspected damage below ground, and uncertainty was felt as to the condition of piles driven on such sites. No information on the distribution of stress in the pile was furnished by the current static pile-driving theory, since this assumed uniform stress-distribution throughout the pile.

The first stage of the work terminated in 1935, when a Paper was presented to The Institution,[2] in which a comprehensive theory of the stress phenomena was described. The analysis took account of the essential wave-character of the stresses, and was shown to be in agreement with the experimental side of the investigation which involved the recording of the stresses in driven piles, the examination of the properties of head packings, and of the behaviour of concrete under impact. A summary of the investigation, accompanied by working charts for the guidance of the engineer in avoiding damage to the piles, was published in the Journal for April, 1936.[3]

In January, 1936, The Institution took a financial interest in this research and formed a joint Sub-Committee with the Federation of Civil Engineering Contractors and the Building Research Station.

Certain practical suggestions for the improvement of pile-driving practice were made in the 1935 Paper,[2] and it was decided that

[1] "Report on the First International Conference on Soil Mechanics and Foundation Engineering." Journal Inst. C.E., vol. 4 (1936–37), p. 135. (November 1936.)

"The Shearing Resistance of Soils." Journal Inst. C.E., vol. 3 (1935–36), p. 333. (June 1936.)

[2] W. H. Glanville, G. Grime, and W. H. Davies, "The Behaviour of Reinforced-Concrete Piles During Driving." Journal Inst. C.E., vol. 1 (1935–36), p. 150. (December 1935.)

[3] Journal Inst. C.E., vol. 2 (1935–36), p. 587. (April 1936.)

further work should, in the first place, be concerned with the production of two suggested devices, an improved head-packing, and an instrument for the measurement of head-stress.

It has been shown that the head-stress, and, in hard driving, the set, were largely dependent on the type and condition of the head-packing and in particular on its stiffness. An improved head-packing, of known and constant characteristics, would therefore enable the stresses in the pile to be controlled and would eliminate one unknown factor in bearing-capacity tests. The difficulty was to find a packing which did not consolidate during driving, but retained a reasonably low stiffness after repeated use. Asbestos packing gave somewhat promising results, but it was felt that a mechanical device would afford the most satisfactory solution of the problem. A hydraulic buffer to replace the helmet and packing has now been designed, and a scale model constructed. The form of the pressure curve is governed by the passage of oil through an orifice, whose shape has been calculated to give a constant stress in the pile-head. No information on the behaviour of such a device under impact was available, and the design had therefore to be checked by comparing records of the oil-pressure and stress in the pile-head. These tests are now being made with a 10-foot model pile.

A simple device for the measurement of head-stresses was described in the report of the main investigation. Work has been continued on this instrument to enable it to be used with existing methods of pile driving. It is hoped ultimately to carry out research on the bearing power of piles, but this aspect awaits the satisfactory solution of the more urgent driving problems.

Velocity Formulas.

The Sub-Committee on Velocity Formulas was formed as a result of a request from the International Executive Council of the World Power Conference for an opinion on the subject of velocity formulas. A Sub-Committee was formed in January, 1936, to investigate Velocity Formulas for Open Channels and Pipes. The following Panels were appointed :—

(1) A Panel to evolve a basic formula universally applicable to pipes and channels and to suggest research where information is required, and

(2) A Panel to collect information with regard to channels and the effect of silt in suspension on the velocity of flow.

The decision to form this Sub-Committee was largely due to the new method of approach to the problem initiated by the researches

of Prandtl and Nikuradse, and to the feeling that this work held promise of useful results for the practical engineer. Dr. C. M. White, a member of the Sub-Committee, had already been engaged in this research for several years, and work on the flow through artificially-roughened pipes and on the flow through channels was already under way at the time of its formation. These researches have been kindly placed at the disposal of the Sub-Committee.

A definite stage in the research was marked by the publication by the Royal Society of a Paper, "Experiments with Fluid Friction in Roughened Pipes," by C. F. Colebrook and C. M. White.[1] Nikuradse, experimenting with pipes in which the roughnesses were of uniform size and closely spaced, found a comparatively abrupt transition from the "smooth" law (where the roughness of the surface has no effect on the flow) at slow speeds to the "rough" law (where the resistance depends upon the surface texture and is proportional to the square of the velocity) at high speeds. Surfaces of the nature of cast iron, wrought iron, or galvanized steel give results which indicate a much more gradual transition between the two resistance laws. These experiments show that with non-uniform roughness, transition is gradual, and in most cases so gradual that the whole working range lies within the transition zone. The resistance of a pipe is therefore a function not only of the size of the elements of roughness but also of their distribution. Consideration has also been given by the same Authors to the reduction of the carrying capacity of pipes with age. In a Paper published by The Institution [2] on this subject a theory is outlined, based upon the assumption that the size of roughness irregularities increases in direct proportion with time. The resulting formula for discharge is found to be not inconsistent with published experimental data. The Basic Formula Panel is now engaged in considering this work, and it is hoped to issue the results in the near future in the form of an interim report.

In connexion with this research a questionnaire was issued to water-supply authorities, catchment boards, consulting engineers and others, seeking information correlating discharge with gradient. The results have been very disappointing, but this is not surprising in view of the small gradient concerned in the larger mains and channels, and complicating factors, such as bends, junctions, and variations in cross section. Reliance has therefore had to be placed on published results of observations with, in addition, some useful

[1] Proc. Roy. Soc. A., vol. 161 (1938), p. 367.

[2] C. F. Colebrook and C. M. White, "The Reduction of Carrying Capacity of Pipes with Age." Journal Inst. C.E., vol. 7 (1937–38), p. 99. (November 1937.)

observations obtained by one of the members of the Sub-Committee, Mr. W. N. McClean.

The Channels Panel have been engaged in testing various formulas against such experimental data as can be obtained from various sources. In addition, evidence has been given before the Panel by eminent hydraulic engineers from India, Egypt, and the Sudan. Velocity formulas such as Manning's and others of a similar nature are in the main derived from canals of smooth and regular section, and their application to natural streams is doubtful. With the multiplicity of roughness occurring in practice it seems also necessary to consider the roughness density, or the number of roughness elements per unit area. The problem is also complicated by the entrainment and transport of solids of various degrees of comminution and density for which no generally recognized law exists.

Wave-Pressures.

Owing to the lack of reliable information with regard to pressures exerted by waves on breakwaters and other similar structures, the International Navigation Congress held in Cairo in 1926 decided to form an International Committee for the purpose of investigating this subject. This Committee had representatives from France, Spain, Italy, and Great Britain, the Chairman of the present Research Committee's Sub-Committee being the British representative.

At the meeting of the International Committee on 12th March, 1928, in Paris, it was decided that each country represented on this Committee should study the question and carry out investigations. In France and in Italy a considerable amount of work has been done in this connexion and very complete self-recording instruments have been erected on the breakwater at Genoa. A further meeting of the International Committee was held at Genoa in June, 1935, when the results obtained there were discussed.

Nothing has so far been done in Great Britain although some years ago proposals put forward by the Admiralty to erect recording instruments at Peterhead were discussed with the Department of Scientific and Industrial Research who were willing to make a grant if satisfactory instruments could be acquired; the opinion was, however, expressed by the late Sir Alfred Ewing that no records of any value would be obtained unless the instrument employed was capable of recording pressures down to at least $\frac{1}{100}$ second duration, and as the only instruments available of this nature at the time would have required an expert to use them, it was considered that the expense entailed would not justify the adoption of the proposal.

Recent work, however, has made possible the recording of pressures down to $\frac{1}{1000}$ second duration, and as a result, it was decided in May, 1935, to appoint a Sub-Committee to study the question.

A closer investigation of the original proposal to place recording instruments at Peterhead revealed great difficulties and disadvantages in siting the experiments at this place. The general advance in the technique of model experiments, and in particular a preliminary experiment carried out at the City and Guilds College, indicated the probable value of this line of approach. It was finally decided to conduct a series of preliminary experiments over a period of 1 year. If the pressures recorded in the model can be correlated with the pressures obtained on a sea wall with waves of medium height, it should be possible to infer from the model the pressures corresponding to extreme wave-conditions. Already much information has been obtained of the nature of the breaking waves causing maximum impact with the help of slow-motion cinematograph records taken through a glass-sided tank.

The work is being administered by the Director of Building Research and is being carried out by Major R. A. Bagnold, under the supervision of Dr. C. M. White, at the City and Guilds College.

The International Committee mentioned above is shortly to issue an interim report recording what has been done to date in France and Italy, and it is hoped that as the investigations proceed it will be possible to exchange information as to the progress of experiments.

Special Cements for Large Dams.

A problem of importance to engineers engaged in the construction of large concrete masses such as dams, is the reduction of shrinkage cracks to the minimum possible. Whilst good design and construction, combined with the use of contraction-joints suitably disposed, will reduce the incidence of cracks, it does not eliminate them under modern conditions of rapid construction. The major cause of crack formation is now usually considered to be the thermal contraction of the mass as it cools from the relatively high temperatures produced within it by the heat evolved from the cement during hardening. An additional problem which has given rise abroad to serious deterioration of concrete dams is the leaching of lime from the set cement in the concrete by very soft waters such as are often found in mountain areas.

At the First International Congress on Large Dams, held in Stockholm in 1933, attention was directed to the need for cements of special properties for use in concrete dams, stress being laid in particular on a low heat-evolution during hardening. The tests

used in normal cement specifications afford no indication of this property, nor of some others of importance, and a primary need was the development of suitable test methods. An International Sub-Committee on Special Cements was formed at the Stockholm Congress to provide a means whereby experience in different countries could be pooled. Following its formation, a British Sub-Committee on Special Cements was set up in May, 1934, as a sub-committee of the British Committee on Large Dams. The purpose for which it was set up was the investigation of methods of testing certain properties of cements which are of special importance in concrete dams and other large concrete structures.

In January, 1936, the Sub-Committee was reconstituted as a Joint Sub-Committee of The Institution and of the British Committee on Large Dams. The committee is kept in touch with the work of similar committees abroad through the International Sub-Committee on Special Cements of the International Committee on Large Dams, on which it is represented by two of its members.

The original Sub-Committee arranged for investigations to be made at the Building Research Station on the measurement of the heat of hydration of cements and of their relative resistance to leaching by soft waters, such as are found in many moorland and mountain areas. These investigations have now been completed. Subsequently the Joint Sub-Committee arranged for some preliminary tests on the shrinkage cracking of restrained concretes to be made in order to afford some comparison of the relative behaviour in this respect of a normal Portland cement, a Portland blast-furnace cement and a low-heat pozzolanic cement. Comparative tests have also been carried out in collaboration with investigators in France, Germany, Sweden, and the United States on the fineness of cements as measured by modern surface-area methods. Similar comparative tests on the heat-evolution of cements are to be made. The purpose of these tests is to compare the results given by different methods, and the reproducibility of the same method in different laboratories. The Sub-Committee has also under discussion a specification for low-heat cements.

A report of the work of the Sub-Committee [1] was submitted to the Second International Congress on Large Dams, held in Washington in September, 1936, and a summary of the present position with regard to special cements for dams was published in the Journal.[2]

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[1] Journal Inst. C.E., vol. 2 (1935–36), p. 175. (February 1936.)
[2] Journal Inst. C.E., vol. 5 (1936–37), p. 217. (February 1937.)

Fish-Passes.

The maintenance of fish life in rivers presents a problem of some importance where the passage of fish upstream is obstructed by dams and weirs, in view of the fact that certain fish breed in the upper reaches of rivers. Some form of fish-pass has to be provided. Such structures are often costly to construct and, in addition, consume a considerable amount of water which would otherwise be conserved. When such wastage of water is not already necessary as compensation water, it represents a valuable and continuous loss.

A Sub-Committee was accordingly formed in December, 1935, to carry out investigations to determine efficient forms of fish-pass for any given water flow and the minimum quantity of water necessary for any form. A scheme of research was prepared and arrangements made for the experimental work to be carried out by Dr. Paul Nemenyi under the direction of Dr. C. M. White in the civil engineering laboratories of the City and Guilds College.

Many of the problems can be studied by scale-model hydraulic experiments checked by large-scale tests to correlate results with practical conditions. In addition, the research includes a study of existing forms of fish-pass. Passes may be classified as :—

(1) the plain chute type ;
(2) a plain channel with obstructions of various types placed in it to reduce the velocity of the water flow and, possibly, to afford places of shelter for the fish ;
(3) a succession of pools with an overfall or submerged orifice leading from pool to pool.

Essential features are the provision of a certain minimum size of opening between pools or obstructions dependent upon the type of fish, the maintenance of a maximum velocity within the power of the fish to surmount, and the provision of sufficient flow to guide the fish to the entrance of the pass. Apart from this last consideration greater efficiency can be obtained with smaller flows. Results already obtained have indicated that maximum gradient and consequently minimum length of pass and cost of construction can be secured with the narrow, rough-channel type. There are, however, many other factors to be taken into account in considering the choice of type, various advantages and disadvantages being inherent in each. The research is now approaching completion and the preparation of a report is in hand.

Reinforced-Concrete Structures for the Storage of Liquids.

In the Report of the Reinforced Concrete Structures Committee of the Building Research Board, issued in 1934, recommendations were made for a Code of Practice for the Use of Reinforced Concrete in Buildings; structures for the storage of fluids were expressly omitted from consideration.

In normal reinforced-concrete design members are proportioned on the assumption that there is no tensile stress in the concrete. This presupposes that the concrete will be unable to resist the tensile stresses set up and that hair cracks will be formed. Whilst this is permissible in normal construction, it would lead to leakage in the case of structures for the storage of liquids, with resulting unsightliness and possible deterioration through corrosion of reinforcement, leaching-out of lime from the concrete, or frost action. Furthermore, in liquid-retaining structures special regard must be paid to the proportioning of the concrete so as to ensure its impermeability.

The lack of information and the need for guidance in respect of such structures led to the formation in June, 1935, of a Sub-Committee to investigate their design and construction and draw up recommendations thereon. Panels were formed to consider the questions of Materials, Design, Construction, and General Matters respectively. The work is now well advanced although some details of design are still under consideration.

The tendency to form cracks as a result of drying-shrinkage and the prevention of such cracking by suitable design are matters concerning which little information is available. In the hope of collecting data on the subject a questionnaire was addressed to certain experts on reinforced-concrete construction. The replies revealed a surprising lack of unanimity regarding the occurrence of such cracking and the practice concerning the spacing of joints to prevent or minimize it.

The efficiency of joints in preventing cracking depends upon the extent to which they relieve constraint, and provision must therefore be made for either sliding or to-and-fro movement at a joint, as the case may be. In order to ensure watertightness under these conditions bituminous jointing materials are commonly used. Little information is available concerning the behaviour of bituminous jointing material under conditions of alternating tension and compression and under alternating shearing force, and a research restricted to a study of its mechanical behaviour has been undertaken for the Sub-Committee by Professor R. G. H. Clements at the City and Guilds College.

Simply Supported Steel Bridges.

The recently completed revision by the British Standards Institution of B.S.S. No. 153 Girder Bridges, Part 3, Loads and Stresses, made evident the need for guidance to engineers on such questions as impact allowances, lurching of locomotives, and other dynamic effects. The B.S.I. revision committee found that whilst the provisional formulas for impact allowance given in the 1923 issue of the specification could not be allowed to remain, agreement had not been reached on any one method to be used in its place.

The Institution of Civil Engineers and the Institution of Structural Engineers decided that it devolved upon them to investigate such matters, and a Joint Committee, the appointment of which was welcomed by the British Standards Institution, was formed to devise a code of practice and advice in connexion with the design and construction of Simply Supported Steel Bridges. The inquiry was limited to simply supported bridges as it was felt that no general code could be formulated for more complex structures, which should receive individual consideration.

The committee is investigating existing practice in various countries and is at the same time taking into account recent developments in materials (such as the increasing use of high-tensile steel) and in design. An attempt is being made to place design on a rational basis so that theoretical requirements shall not be violated.

Wind-Pressure on Bridges.—In considering wind loads the committee decided that there was insufficient information directly applicable to bridges. The work of Stanton and subsequent research does not cover the effect of the shape of cross section of members, shielding of leeward girders, the effect of bridge-flooring, and the obliquity of the wind. It was accordingly decided to institute a research into wind-pressures on simply supported bridges. The Department of Scientific and Industrial Research is assisting, both financially and by arranging for wind-tunnel tests to be carried out in the 7-foot wind-tunnel at the National Physical Laboratory.

A scheme of research has been drawn up covering both plate- and lattice-girder construction, and a start on the tests has been made. In the case of the lattice-girder construction an investigation is being made in the first place into the possibility of replacing "true-to-scale" model members by conventional members of equivalent rectangular section. It should then be possible to investigate the effects on the total pressure of the transparency-ratio, the spacing of main girders, the type of floor-construction, the presence of a train on the bridge, and other factors, using models with simple conventional members.

Steel Structures.

In April, 1936, the Final Report of the Steel Structures Research Committee was issued. As the result of their investigations this Committee proposed radical changes in the procedure of design and practice in respect of the magnitude and distribution of bending moments as between columns and beams. It was considered, however, that before it could be hoped to bring the new methods into general use, they must be simplified and condensed into convenient rules for design. Accordingly in March, 1937, a Joint Committee was formed of the Institutions of Civil and Structural Engineers to formulate a code of practice for the use of structural steelwork in buildings, based upon the Final Report of the Steel Structures Research Committee.

Repeated Stresses in Structural Elements.

Since Sir William Fairbairn, in the middle of the nineteenth century, carried out experiments on the effect of slowly-repeated loadings on a built-up wrought-iron girder, little experimental work has been done on the effect of repeated stresses on fabricated elements. The classic work of Wöhler was carried out on machined specimens, and during recent years a very large amount of work has been done in Great Britain and other countries on machined specimens of various metals. The effect of atmospheric and corrosive conditions on the fatigue of metals under stress has also received a considerable amount of attention. Mathematical analysis of certain simple cases and experimental investigation by optical and other means have shown that, under elastic conditions, considerable stress-concentrations may occur at discontinuities.

It has also been shown that the fatigue resistance of steels that have been quenched and tempered is very much lower in the "black" condition than after machining. It appeared to Professor F. C. Lea that it was, therefore, a matter of considerable practical importance to continue the work of Fairbairn, and he instituted a research to determine the fatigue ranges of rolled sections such as are used in structural work, and to investigate the effect thereon of riveted and welded joints.

Certain preliminary experiments were made on black plates and on small riveted joints and bolted joints under repeated tensile stresses.[1] In order to carry out experiments on unmachined welds

[1] F. C. Lea, "Repeated Stresses on Structural Elements." Journal Inst. C.E., vol. 4 (1936–37), p. 93. (November 1936.)

of plates and on various types of riveted and welded joints, used or proposed in structural work, it was necessary to design special machines. The preliminary experiments on a machine designed to test under bending rolled-steel joists up to a depth of 5 inches indicated that the resistance to repeated stresses of riveted and welded joints was much less than that of the original beam.[1]

In view of the significance of these results The Institution and the Department of Scientific and Industrial Research decided to give financial assistance to Professor Lea to enable him to carry out further work. A Sub-Committee of the Research Committee was formed to follow the progress of the research and assist in a consultative capacity. Considerable assistance in the provision of fabricated specimens was obtained from industrial firms.

The programme of research consists of a main series of repeated bending-tests of rolled girders containing joints of various types, together with subsidiary static bending-tests and tensile, impact, hardness and bending-tests on specimens cut from the girders. The following series of girders were first prepared for test :—

B.S. 5-inch by 3-inch by 11-lb., as-rolled joists, with rivet holes in the tension flange.
,, ,, ,, with a butt-welded joint at the centre.
,, ,, ,, with a fillet-welded cover-plate joint at the centre.
,, ,, ,, with a riveted cover-plate joint at the centre.
Built-up welded girders 5 inches by 3 inches, consisting of a web fillet-welded to the flange plates.
Similar girders after heat-treatment at 650° C.

Tests on the above series have been completed. These tests have shown the very great importance of the design and proper fabrication of welds, and particularly the significance of interpenetration of weld- and parent-metal if safe repeated stresses are to be reasonably high.

Specimens have been or are being prepared for further tests as follows :—

(1) To ascertain the effect of multi-run as against single-run fillets.

[1] F. C. Lea and J. G. Whitman, "The Failure of Girders under Repeated Stresses." Journal Inst. C.E., vol. 7 (1937–38), p. 119. (November 1937.)

(2) For a series of tests using high-tensile structural steel (fillet-welds).
(3) For a series with the flanges similar to short-stalked T-sections so that the webs may be butt-welded.
(4) For a series using two different types of electrodes, laying respectively "smooth" and comparatively "rough surface" fillets.
(5) For a series with the sections welded to a central plate to simulate the effect of welded end plates or the direct welding of a beam to a stanchion.

Consideration is being given to other types of joints and to other materials than carbon steels.

Earthing to Metal Water-Pipes and Mains.

The Electricity Acts, 1882 and 1933, and the Electricity Supply Regulations arising therefrom, provide for the necessity of earthing a direct- or alternating-current system of transmission and distribution, and also the metal-work enclosing or supporting electrical apparatus. The Factory Acts provide for similar and stringent regulations for the safety of the operative in factories. Telegraph, telephone and wireless apparatus also necessitate in most cases a connexion to earth, and it has been the custom for years in domestic and other premises to earth electrical apparatus, conduits, and wireless installations to the incoming water-services where such service exists. This forms the cheapest and best method, and usually provides for a path of low resistance to earth.

Many cases have arisen where corrosion of water-mains has been attributed to such earthing, and there have been instances where electric shocks have been incurred during the repair of water-mains. Whilst it is generally acknowledged that continuous current causes electrolytic corrosion where it leaves a metal conductor or pipe to earth, there is some doubt about the action of alternating current.

In view of the importance of the subject to both electrical engineers and water engineers, a Sub-Committee on Earthing to Metal Water-Pipes and Mains was set up with the approval and collaboration of the Institution of Electrical Engineers, the Institution of Water Engineers, the British Waterworks Association, and the Water Companies' Association, with the following terms of reference :—

"To explore the problem of possible injury to metal water-pipes and mains through the earthing thereto of electrical installations, particularly in relation to alternating currents, with a view

(1) to investigating the existence and extent of such injury, research being carried out if necessary;

(2) to obtaining mutual agreement on the conditions under which earthing connexions to water-pipes and mains might be made; and

(3) to formulating, if necessary, a set of regulations in respect thereof."

A Drafting Panel has been successful in reaching agreement on a set of draft regulations, which have now been submitted to the various Institutions and bodies represented on the Sub-Committee for their comments.

In the course of their work, the Sub-Committee came to the conclusion that although they had been able to agree on the draft regulations, some research was nevertheless desirable to ascertain the true facts regarding the possibility of corrosion due to earthing, and arrangements are in hand for the investigation of this question by the British Electrical and Allied Industries Research Association. The research proposed consists of the investigation of:—

(i) The amount and effect of aggregate leakage-currents on water pipes.

(ii) The possibility of partial rectification of alternating currents in underground water-supply systems

  (*a*) at earthing connexions;

  (*b*) between metal pipes and the soil.

(iii) The possibility of primary-cell effects in water-supply systems.

(iv) The relation of the above to the question of corrosion.

The research is estimated to occupy 3 years.

Breathing Apparatus for Use in Sewers.

In 1934 the Ministry of Health issued a Report on Accidents in Sewers with a supplement on Measures for Preventing Accidents in Sewers and Sewage Tanks. It was felt that a useful purpose would be served by a complementary document giving rules for the use of breathing apparatus in sewers, etc., somewhat on the lines of the Breathing Apparatus Order, 1924, for Coal Mines.

A Sub-Committee was formed in November, 1935, with the following terms of reference:—

"To examine and report on the necessity for prescribing practice for the use of self-contained breathing apparatus for use in sewers, tunnels, and similar civil engineering

works. In dealing with the matter the Sub-Committee should have regard to the publications on the subject by the Ministry of Health in 1934."

It was realized at an early stage of the Sub-Committee's deliberations that it would be impracticable and undesirable to specify conditions for tunnels and similar civil engineering works under construction. It was therefore decided that these should be excluded from the purview of the rules.

An early step was to examine existing types of breathing apparatus on the market. It was possible to suggest various modifications which would be improvements for the purpose in view, and Messrs. Siebe, Gorman have co-operated by making the suggested alterations and submitting apparatus for test. Tests have been made both at the Mines Testing Station, Doncaster, and at Birmingham University; work at the latter place, although still in progress, is now nearing completion. It will be possible to specify the essential features of apparatus suitable for rescue work in sewers and confined spaces.

The Sub-Committee proceeded to appoint a Drafting Panel to draw up Recommendations for the Use of Breathing Apparatus in Sewers. This work has been completed and the recommendations approved by the Sub-Committee. Before issuing their report it was decided to await the results of the tests of apparatus, as it is considered desirable that a specification of apparatus should accompany recommendations for its use.

APPENDIX I.

Membership of Committees.

Research Executive Sub-Committee.

Sir Clement D. M. Hindley, K.C.I.E., M.A. (*Chairman*).
W. J. E. Binnie, M.A.
Professor Gilbert Cook, D.Sc.
W. T. Halcrow.
R. G. Hetherington, C.B., O.B.E., M.A.
Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S.
G. G. Lynde.
R. E. Stradling, C.B., M.C., Ph.D., D.Sc.

Sub-Committee on the Effect of Soils containing Sulphate Salts on Concrete and Metal Pipes.

(*June*, 1935, *to December*, 1936.)

R. G. Hetherington, C.B., O.B.E., M.A. (*Chairman*).
W. J. E. Binnie, M.A.
Eric Burt.
J. R. Davidson, C.M.G., M.Sc.
C. H. Desch, D.Sc., F.R.S.
George Ellson, O.B.E.
H. J. Escreet.
A. F. Hallimond, D.Sc.
C. L. Howard Humphreys, T.D.
J. G. Kay.
F. M. Lea, D.Sc.
Alexander Melville (*Since deceased*).
S. G. S. Panisset (*Since deceased*).
J. H. Smith.
R. H. H. Stanger.
J. D. Watson.
F. Wilkinson.
P. H. Wilson.

Committee on Soil Corrosion of Metals and Cement Products.

Sir George Humphreys, K.B.E. (*Chairman*).
D. C. Crichton.
E. M. Crowther, D.Sc.
J. R. Davidson, C.M.G., M.Sc.
C. H. Desch, D.Sc., Ph.D., F.R.S.
S. B. Donkin.
W. H. Hatfield, D.Met., F.R.S.
R. G. Hetherington, C.B., O.B.E., M.A.
J. G. Kay.
Harold Moore, C.B.E., D.Sc.
Professor G. W. Robinson, M.A.
R. E. Stradling, C.B., M.C., Ph.D., D.Sc.
H. G. Taylor, M.Sc.

Sub-Committee on Soil Corrosion of Cement Products.

R. G. Hetherington, C.B., O.B.E., M.A. (*Chairman*).
D. C. Crichton.
E. M. Crowther, D.Sc.
E. A. Dancaster, Ph.D., M.Sc.
C. L. Howard Humphreys, T.D.
J. G. Kay.
F. M. Lea, D.Sc.
Alexander Melville (*Since deceased*).
W. P. Robinson.
R. H. H. Stanger.
G. M. C. Taylor, M.C., M.A.
N. B. Walker.
J. D. Watson.
P. H. Wilson.

Sub-Committee to Collect Existing Information regarding Corrosion of Metals.

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Harold Moore, C.B.E., D.Sc.

Sub-Committee on the Effect of Conditions of Environment on Corrosion.

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Alfred Brandt.
E. M. Crowther, D.Sc.
H. G. Dines.
J. C. Hudson, D.Sc.
F. M. Lea, D.Sc.
H. G. Taylor, M.Sc.

Joint Sub-Committee on Vibrated Concrete with the Institution of Structural Engineers.

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G. M. Burt.
W. L. Cowley.
H. R. Cox.
Robert Ducas.
F. M. G. Du-Plat-Taylor.
Oscar Faber, O.B.E., D.Sc.
W. H. Glanville, D.Sc., Ph.D.
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R. E. Holloway.
W. L. Lowe-Brown, D.Eng., M.Sc.
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William Muirhead.
Stanley Vaughan, B.Sc.
W. H. Woodcock.
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Professor W. N. Thomas, M.A., D.Phil., M.Sc.
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J. S. Wilson.

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W. L. Cowley.
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G. B. R. Pimm (*Since resigned*).
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R. E. Stradling, C.B., M.C., Ph.D., D.Sc.

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B. W. Bryan.
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Gerald Lacey, B.Sc.
J. M. Lacey.
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R. W. S. Thompson, B.Sc.
C. M. White, B.Sc., Ph.D.

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H. H. G. Mitchell, O.B.E.
R. E. Stradling, C.B., M.C., Ph.D., D.Sc.
C. M. White, B.Sc., Ph.D.
F. B. Young, O.B.E., D.Sc.

Joint Sub-Committee on Special Cements with The British Committee on Large Dams of the World Power Conference.

R. E. Stradling, C.B., M.C., Ph.D., D.Sc. (*Chairman*).
D. C. Crichton.
J. S. Dunn, M.A., Ph.D.
R. G. Franklin.
W. T. Halcrow.
B. M. Hellstrom.
F. M. Lea, D.Sc.
S. G. S. Panisset (*Since deceased*).
R. H. H. Stanger.
W. H. Woodcock.

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W. T. Halcrow (*Chairman*).
Reginald Beddington.
Rustat Blake, M.A.
W. F. H. Creber.
T. E. Hawksley, B.A.
David Lyell, C.M.G., C.B.E., D.S.O.
A. M. MacTaggart.
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W. J. M. Menzies.
T. E. Pryce-Tannatt.
J. C. A. Roseveare.
C. M. White, B.Sc., Ph.D.

Sub-Committee on Reinforced Concrete Structures for the Storage of Liquids.

W. T. Halcrow (*Chairman*).
Professor Cyril Batho, D.Sc., B.Eng.
H. J. Deane, B.E.
Oscar Faber, O.B.E., D.Sc.
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F. O. Kirby, M.Sc.
Sidney Little.
H. B. Milner, M.A.
H. C. Ritchie.
W. L. Scott.

Joint Committee on Simply-Supported Steel Bridges with The Institution of Structural Engineers.

Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S. (*Chairman*).
David Anderson, B.Sc., LL.D.
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B. L. Hurst.
R. Travers Morgan, M.Eng.
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Professor A. J. S. Pippard, M.B.E., D.Sc.
R. H. Thomason.

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David Anderson, B.Sc., LL.D.
H. J. Gough, M.B.E., D.Sc., F.R.S.
Professor F. C. Lea, O.B.E., D.Sc.
Professor A. J. S. Pippard, M.B.E., D.Sc.
Professor Andrew Robertson, D.Sc.

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H. F. Cronin, M.C., B.Sc.
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R. W. James.
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F. W. Purse.
J. D. K. Restler, O.B.E.
P. J. Ridd.
H. W. Swann.

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R. G. Hetherington, C.B., O.B.E., M.A.
J. B. L. Meek.
E. J. Messent.
Professor K. Neville Moss, O.B.E., M.Sc.

APPENDIX II.

List of Bodies which have Given, or Promised, Assistance in connexion with the Researches.

Research.	*Contributors.*
Soil Corrosion of Cement Products	Department of Scientific and Industrial Research (Building Research Station). Cement and Concrete Association. Ellis (per James Smith Bros.). Johnson Bros. (Contractors), Ltd. Lafarge Aluminous Cement Co., Ltd. Melville, Dundas & Whitson. Mono Concrete Co., Ltd. Simpson (Rye Harbour), Ltd. South Coast Cast Concrete Co. Stanton Ironworks Co., Ltd. Trollope & Colls, Ltd. Turners Asbestos Cement Co. Ure and Menzies.
Vibrated Concrete	Institution of Structural Engineers. Department of Scientific and Industrial Research (Building Research Station). Cement and Concrete Association. Federation of Civil Engineering Contractors. Federation of Manufacturers of Artificial Stone. Imperial Chemical Industries, Ltd. Institute of Builders. Lafarge Aluminous Cement Co., Ltd. Reinforced Concrete Association.
Earth-Pressures	Department of Scientific and Industrial Research (Building Research Station).
Pile Driving	Department of Scientific and Industrial Research (Building Research Station). Federation of Civil Engineering Contractors.
Wave-Pressures	Department of Scientific and Industrial Research (Building Research Station).
Special Cements	Department of Scientific and Industrial Research (Building Research Station). British Committee on Large Dams. Association of Portland Cement Manufacturers. Caledonian Portland Cement Co. Casebourne & Co., Ltd. Cement and Concrete Association. Federation of Civil Engineering Contractors. Institution of Structural Engineers. Ketton Portland Cement Co. Tunnel Portland Cement Co.

Fish-Passes	The Development Commissioners. British Aluminium Co., Ltd. City and Guilds College. Grampian Electricity Supply Co. Hon. Fishmongers' Co.
Reinforced-Concrete Structures for the Storage of Liquids.	Professor R. G. H. Clements, M.C.
Simply Supported Steel Bridges .	Institution of Structural Engineers. Department of Scientific and Industrial Research (National Physical Laboratory).
Repeated Stresses in Structural Elements.	University of Sheffield. Appleby-Frodingham Co.
Earthing to Metal Water-Pipes and Mains.	British Electrical and Allied Industries Research Association. British Waterworks Association. Institution of Electrical Engineers. Water Companies Association.
Breathing Apparatus for Use in Sewers.	University of Birmingham. Mines Rescue Station, Doncaster. Siebe, Gorman & Co., Ltd.

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